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THIRD PROGRESS REPORT
INVESTIGATION OF PRESTRESSED CONCRETE FOR HIGHWAY BRIDGES

CONFERENCE ROOM

JUNE, 1954

ENGINEERING EXPERIMENT STATION
UNIVERSITY OF ILLINOIS
URBANA, ILLINOIS

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THIRD PROGRESS REPORT
OF THE
INVESTIGATION OF PRESTRESSED
CONCRETE FOR HIGHWAY BRIDGES

Conducted by

THE ENGINEERING EXPERIMENT STATION

UNIVERSITY OF ILLINOIS

In Cooperation with

THE DIVISION OF HIGHWAYS

STATE OF ILLINOIS

and

THE BUREAU OF PUBLIC ROADS

U. S. DEPARTMENT OF COMMERCE

June 1954

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A. SUMMARY OF ACTIVITIES

1. History of Project

This investigation of prestressed concrete for highway bridges was begun in the summer of 1951 as a cooperative project of the University of Illinois Engineering Experiment Station and the Illinois Division of Highways with participation by the U. S. Bureau of Public Roads through Federal Aid.

Two previous progress reports have covered the work done up to May 1953. This report summarizes the activities of the project during the period May 1953 through May 1954.

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2. Organization

The work on this investigation has been carried out in the Structural Research Laboratory of the Department of Civil Engineering under the general supervision of N. M. Newmark, Research Professor of Structural Engineering, and C. P. Siess, Research Associate Professor of Civil Engineering.

Guidance to the project staff has been provided by an Advisory Committee including representatives from each of the cooperating agencies. During the past year, the membership of the Advisory Committee has been as follows:

Representing the Illinois Division of Highways:

W. E. Chastain, Sr., Engineer of Physical Research,
Bureau of Research and Planning

W. J. Mackay, Bridges and Traffic Structures Section, Bureau of Design

C. E. Thunman, Bridges and Traffic Structures Section, Bureau of Design

Representing the Bureau of Public Roads:

R. Archibald, Division Engineer, Division 7

E. L. Erickson, Chief, Bridge Branch

E. F. Kelley, Chief, Physical Research Branch

Representing the University of Illinois:

N. M. Newmark, Chairman

C. P. Siess

I. M. Viest, Research Assistant Professor of Theoretical and Applied Mechanics

The work of the project has been under the immediate supervision of J. H. Appleton, Research Associate in Civil Engineering. The remaining project personnel are all graduate students in Civil Engineering, devoting from one-half to full time to research on this project. They are A. Feldman, P. C. Gardner, N. A. Gouvis, G. McLean, M. A. Sozen, and J. E. Villepelet, all Research Assistants. In addition, F. R. Khan, graduate student in Civil Engineering; and K. Okada, Assistant Professor, Kyoto University, Kyoto, Japan, and a graduate student in Civil Engineering, gave valuable assistance to various phases of this investigation.

3. Scope of Work

The work completed since the beginning of the investigation and that now underway can be divided into five categories as follows:

(a) Studies of beams failing in flexure.

(b) Studies of beams failing in shear.

(c) Studies of time-dependent effects due to creep, shrinkage, and relaxation.

(d) Studies of current literature.

(e) Related studies.

The work categories are discussed in the following five sections. In each section the work accomplished during the past year in each category is summarized and brief mention is made of the previous work of the investigation.

4. Studies of Beams Failing in Flexure

This phase of the project has been carried out by Mr. Feldman with the assistance of Mr. Villepelet, and the results are reported fully in the thesis by Mr. Feldman which has been distributed separately from this Progress Report. The thesis covers the work completed this year on bonded and unbonded rectangular beams.

(a) Bonded, Post-tensioned, Rectangular Beams: Tests were made on six bonded, post-tensioned, rectangular beams reinforced with steels having stress-strain characteristics widely different from those previously used on this phase of the investigation. The object of these

tests was to further check the accuracy of the theory for computing the ultimate flexural strength presented in Billet's thesis, which was issued as a part of the Second Progress Report. In general, the agreement between measured and computed phenomena was good and confirmed the belief that variations in stress-strain properties of the steel would not affect the applicability of the theory to the computation of flexural capacity and deformations.

These tests represent an extension of the tests of 27 similar beams reported in Billet's thesis. Thus the variables covered in tests of bonded, rectangular beams now include concrete strength, steel ratio, tensioning stress, and stress-strain properties of the reinforcement.

(b) Unbonded, Post-tensioned, Rectangular Beams: Tests were made on 15 unbonded, post-tensioned, rectangular beams. The object of these tests was to study the behavior of unbonded prestressed concrete beams. On the basis of the test results, an analysis for computing the flexural strength of unbonded beams was developed by modifying the theory for bonded beams in a manner similar to the modification presented in the First Progress Report. This study must be considered a preliminary one in that a full range of variables was not covered. Further tests and studies should serve to check the accuracy of the theory and extend its range of applicability.

(c) Miscellaneous: One study has been completed by Mr. F. R. Khan and Mr. Appleton which involves a comparison of measured and computed moments of beams tested elsewhere. This study was conducted

in order to check the applicability of the analysis presented in the First Progress Report and in Billet's thesis to full-size beams with various shapes and methods of prestressing. This study is reported in this Progress Report.

5. Studies of Beams Failing in Shear

This phase of the project was carried out by Mr. Gardner with the assistance of Mr. Sozen until November 1953, and from that date by Mr. Sozen. A report on the results of this phase of the project is included in this Progress Report and covers the work completed this year on rectangular beams and I-beams.

(a) Bonded, Post-tensioned, Rectangular Beams: Tests were made on five post-tensioned, rectangular beams without web reinforcement, designed to fail in shear or near the boundary between shear and flexural failures. The objects of these tests were to establish a limit between shear and flexural failures for beams of this type with zero prestress and to make a check test on one beam tested by E. M. Zwoyer.

These tests represent an extension of the tests of 34 similar beams reported in Zwoyer's thesis. Thus the variables covered in tests of bonded, rectangular beams failing in shear now include concrete strength, steel ratio,shear span, and tensioning stresses from zero to 150,000 psi.

(b) Bonded, Pre-tensioned, I-beams: Tests were made on 26 pre-tensioned I-beams failing in shear. In addition to the experimental work, analytical studies of the results have been made. The tests indicate that the manner of failure of I-beams in shear is somewhat similar to that of rectangular beams. The concept of beams failing at a limiting shear moment, derived from tests on rectangular beams, seems to hold also for I-beams.

Tests of four I-beams were reported in Zwoyer's thesis, and these tests should be considered a part of the present study of the shearing strength of I-beams.

6. Studies of Time-Dependent Effects in Prestressed Concrete

This phase of the investigation was carried out by Mr. McLean with assistance of Mr. Okada in the analytical phase of the program. The results of this study are reported fully in the thesis by Mr. McLean which has been distributed separately from this Progress Report and in a short report included in this Progress Report. The thesis covers the work completed on this phase of the project during the past year, with the exception of an analytical study of the deformations of test beams under sustained loads reported in this Progress Report, and also that reported in the Second Progress Report. The work completed this year includes a further study of the literature on relaxation, creep, and shrinkage; relaxation tests on high tensile strength wires; the development of an analytical procedure for evaluating the combined effects of creep, shrinkage, and relaxation on prestressed concrete beams; and analytical studies of the deformations of test beams under sustained loads.

(a) Studies of the Literature on Relaxation, Creep, and Shrinkage: A continuing study of the literature on relaxation, creep, and shrinkage has been made relative to the effects to be expected on prestressed concrete beams. These studies reported in McLean's thesis include those reported in the Second Progress Report plus items abstracted during the current year. A summary of tests made elsewhere is presented in as uniform a fashion as possible.

(b) Relaxation Tests on High Tensile Strength Wire: The relaxation tests on high tensile strength wires have been conducted not only with the object of obtaining relaxation characteristics of various wires but also with the object of developing a relatively convenient test procedure for determining relaxation characteristics. The tests reported in McLean's thesis include those reported in the Second Progress Report and a large number of tests conducted this year. The tests include relaxation tests on various types of wires and tests on the effects of overstressing on relaxation.

(c) Analytical Procedure for Evaluating the Combined Effects of Creep, Shrinkage, and Relaxation on Prestressed Concrete Beams: An analytical procedure for evaluating the combined effects of creep, shrinkage, and relaxation on prestressed concrete beams is presented in McLean's thesis. Certain assumptions relative to the creep-time relationship have made possible the development of a relatively simple expression for the losses of prestress in a prestressed concrete beam. To check these assumptions and test the analytical procedure, a series of sustained load tests on beams and cylinders has been initiated.

(d) Analytical Studies of the Deformations of Test Beams Under Sustained Loads: In connection with the work preceding the initiation of sustained load tests on prestressed concrete beams, some theoretical computations involving losses and deformations were made regarding the losses to be expected for the test beams. These computations were thought to be of interest in that they constitute an application of the analytical procedure presented in McLean's thesis. Therefore, a short report on these studies is included in this progress report.

7. Studies of Current Literature

During the past year, some of the current literature was abstracted for use in connection with the other phases of the investigation. These items are included as references in the thesis issued as parts of this Progress Report. Since the American Concrete Institute will publish a very comprehensive bibliography on prestressed concrete, it is contemplated that no further bibliographies like those issued in the First and Second Progress Report will be undertaken. However, literature studied in connection with the various phases of the investigation will be listed as references to the regular reports.

8. Related Studies

Other studies of prestressed concrete which are not part of this investigation but are concerned with related problems are in progress at the University of Illinois.

Mr. F. Khan, a full-time graduate student in Civil Engineering has begun a Ph. D. thesis which will involve analytical studies of the relation between working stress and ultimate strength criteria for prestressed concrete. Although not a part of this investigation, this thesis is being directed by Professor Siess, and it is expected that the results will be available for use in connection with the work of the investigation.

In addition, various staff members of the Department of Civil Engineering and Theoretical and Applied Mechanics are serving as task committee members for the preparation of a report on recommended practice for prestressed concrete (ACI-ASCE Committee 323). Although their activities on this committee are not directly related to this investigation, the experience they gain in attempting to formulate rules for design and construction will be available for use in connection with the investigation.

9. Future Program

The work planned for the coming year is as follows:

(a) Beams in Flexure: An extension of the work on unbonded beams started by Mr. Feldman and reported by him in a thesis will be continued to cover more variables.

(b) Beams in Shear: An extension of the work on I-beams being conducted by Mr. Sozen will be continued. In addition, it is expected that a start can be made on tests of beams with web reinforcement.

(c) Time-Dependent Effects: The relaxation tests on high tensile strength wires will be continued. Sustained load tests on beams will be continued. Analytical studies relative to the effects of creep, shrinkage, and relaxation will be continued.

This program for the coming year follows the outline of proposed tests approved by the Advisory Committee and summarized in the Supplement to the Minutes of the Meeting of October 29, 1953.

At various times during the course of the investigation, a number of items have been proposed or suggested for further study. These items are summarized along with a few others in the following list and are not necessarily in any specific order.

Flexure

Bonded Beams
- Beams with center loading.
- Beams with T or I cross-sections.
- Beams with compression reinforcement.
- Continuous beams.
- Slabs.

Unbonded Beams
- Effect of varying prestress.
- Effect of large L/d ratios on beam strength.
- Beams with bonded mild-steel reinforcement in the bottom of the beam.

Shear

I-beams
- Beams with web reinforcement, either curved cables or stirrups.

Rectangular beams.
- Beams with web reinforcement, either curved cables or stirrups.

Miscellaneous

Beams assembled from precast units.
End anchorages - strength, efficiency, fatigue, ...
Bearing stresses on end blocks.
Notch sensitivity.
Stress corrosion.
Composite construction.
Fatigue strength.
Bond.
Pretensioning.

Although these problems are all of current interest, not all of them can be included in the work of the investigation next year or even in the near future. However, several of these problems are being studied elsewhere, and results should be forthcoming. Consequently, the work program for the investigation proposed for the coming year has been based on several factors: (1) programs underway, but not completed, should be followed to completion; (2) programs which are a logical sequence of past programs should be started before other programs involving extensive new studies and equipment; (3) the size of the programs must be selected to fit the requirements of personnel and finances.

10. Reports and Publications

The reports and publications issued during the course of this investigation are listed in chronological order.

"First Progress Report - Investigation of Prestressed Concrete for Highway Bridges," Engineering Experiment Station, Structural Research Series No. S-9, April, 1952.

"Bibliography on Prestressed Concrete", Engineering Experiment Station, Structural Research Series No. S-10, April, 1952.

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"Tests of a Full-Size Prestressed Concrete Block Beam", Reported by J. H. Appleton, Conducted as a part of the Investigation of Prestressed Concrete for Highway Bridges, June, 1952.

"Shear Strength of Simply-Supported Prestressed Concrete Beams", a Thesis by E. M. Zwoyer, Issued as a Part of the Second Progress Report of the Investigation of Prestressed Concrete for Highway Bridges, Civil Engineering Studies, Structural Research Series No. 53, June 1953.

"Study of Prestressed Concrete Beams Failing in Flexure", a Thesis by D. F. Billet, Issued as a Part of the Second Progress Report of the Investigation of Prestressed Concrete for Highway Bridges, Civil Engineering Studies, Structural Research Series No. 54, June, 1953.

"Second Progress Report - Investigation of Prestressed Concrete for Highway Bridges", June, 1953.

"A Study of Time-Dependent Variables in Prestressed Concrete", a Thesis by G. McLean, Issued as a Part of the Third Progress Report of the Investigation of Prestressed Concrete for Highway Bridges, Civil Engineering Studies, Structural Research Series No. 74, June, 1954.

"Bonded and Unbonded Prestressed Concrete Beams Failing in Flexure", a Thesis by A. Feldman, Issued as a Part of the Third Progress Report of the Investigation of Prestressed Concrete for Highway Bridges, Civil Engineering Studies, Structural Research Series No. 75, June, 1954.

"Third Progress Report - Investigation of Prestressed Concrete for Highway Bridges", June, 1954.

"Flexural Strength of Prestressed Concrete Beams", D. F. Billet and J. H. Appleton, Journal of the American Concrete Institute, June, 1954.

"The Ultimate Strength in Shear of Simply-Supported Prestressed Concrete Beams Without Web Reinforcement", E. M. Zwoyer and C. P. Siess, Journal of the American Concrete Institute, scheduled for publication Fall, 1954.

Part B

PROGRESS REPORT

on

A STUDY OF TESTS ON PRESTRESSED CONCRETE BEAMS

by

F. R. Khan and J. H. Appleton

B. A STUDY OF TESTS ON PRESTRESSED CONCRETE BEAMS

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B. A STUDY OF TESTS ON PRESTRESSED CONCRETE BEAMS

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B. A STUDY OF TESTS ON PRESTRESSED CONCRETE BEAMS

I. INTRODUCTION

Most of the investigations of the flexural strength of prestressed concrete beams have been concerned with analyses of rectangular beams and tests on small scale rectangular beams. However, almost all prestressed concrete beams in use are of T or I shape. This difference between research and design has brought up the question as to whether the results of the analyses and tests can be applied to the computation of the flexural strength of bonded full-size prestressed concrete beams.

A number of methods have so far been presented for computing the ultimate flexural strength of prestressed concrete beams of rectangular shape. All of the methods require definite experimental support before they can be used for sections other than rectangular. The primary object of this study is to compare the results of tests on bonded prestressed concrete beams of various sizes and shapes with the results predicted by the analytical procedure presented in two reports of the Investigation of Prestressed Concrete for Highway Bridges (1) (2).*

It was felt that the analysis presented in the two reports is applicable to prestressed concrete beams of all shapes, with some limitations. However, several empirical parameters appear in the analysis. These parameters, which describe the compressive stress block, were evaluated by Billet (2) on the basis of tests on twenty-

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*Numbers in parentheses refer to entries in the Bibliography.

seven rectangular beams, 6 x 12 in. in size. One of the objectives of this study was to determine whether the use of the values of the parameters derived from tests on rectangular beams would lead to a satisfactory correlation between the measured and computed ultimate moments of beams of various shapes and sizes. A satisfactory correlation in itself would not establish the correctness of the individual values of the empirical parameters but would lead to confidence in the applicability of the combination of the parameters to various types of beams.

To attain the objectives of this study, a number of laboratory and field test results, reported in journals since 1940, have been critically studied. The results of 33 beam tests have been compared with moments computed by use of the analysis presented by Billet (2). The beams studied have a large number of variables, and the range of all the different beams may well include most probable prestressed concrete beams and girders that may be commonly used in practice. The spans of the beams vary from 6 ft. 6 in. to 158 ft., the sizes of the beams range from 2 in. by 4 1/2 in. to 52 in. by 70 in., the steel ratio p varies from 0.063 per cent to 1.000 per cent, the concrete strength ranges from 1930 psi to 8200 psi, the steel strength varies from 145,000 psi to 300,000 psi, the initial prestress varies from 0 to 180,000 psi, the parameter Q' varies from 4.3 to 70.0, and the shapes are rectangular, I, T, or box-sections. It is interesting to note that these beams include Freyssinet's, Magnel's and Lee-McCall's system of prestressing, and also Abeles' method of partial prestressing.

The notation and terms used in this study have been defined previously in Billet's thesis (2) and therefore are not repeated here.

II. METHOD OF INVESTIGATION

The ultimate moment of a prestressed concrete beam can be calculated by the general expression

$$M_{ult} = f_{su} A_s jd \qquad (1)$$

where jd is the effective lower arm at failure of the beam.

From the expression above it is obvious that two quantities, f_{su} and jd, must be evaluated for any beam in order to compute its flexural strength. In test reports where the stress-strain properties of the steel, the concrete strength, the effective tensioning stress, the area of tension reinforcement, and the shape of the beam are described in full, the ultimate steel stress f_{su} is computed by use of two relations between steel stress and steel strain. One relation is the stress-strain curve of the steel, and the other is

$$f_{su} = \frac{k_1 k_3 f'_c}{p} \quad \frac{\epsilon_u}{\epsilon_{su} - \epsilon_{se} + \epsilon_u - \epsilon_{ce}} \qquad (2)$$

The method of solution is described in full in Billet's thesis (2).

At this point, it should be noted that the above expression, as well as all the expressions used hereafter, are valid only when the compression zone at failure is rectangular. Consequently, these expressions cannot be used for T, I, and box sections if the neutral axis at failure falls below the flange.

The effective lever arm at failure may be expressed as

$$jd = d(1 - k_2 k_u) \qquad (3)$$

where $$k_u = \frac{f_{su}p}{k_1 k_3 f'_c}$$

Thus, the ultimate moment may be calculated from the expression

$$M_{ult} = f_{su} A_s d(1 - k_2 k_u) \qquad (4)$$

In computations for ultimate moment, the following equation for $k_1 k_3$ given by Billet (2) was used

$$k_1 k_3 = \frac{3000 + \frac{1}{2} f'_c}{1500 + f'_c} \qquad (5)$$

and k_2 was taken as 0.42. In cases where the concrete strength for a beam was given as the cube strength, the corresponding strength for 6 by 12-in. cylinders was taken as the cube strength divided by 1.18.

In all computations ϵ_u was taken to be 0.0034, the average value found for Billet's beams (2).

In a large number of reports on beam tests, the data are very limited. In such cases, certain assumptions must be made regarding the missing data in order to solve for the ultimate steel stress. In many of the test reports, the tensile strength of the steel was given, but not the stress-strain curve. Reasonable existing stress-strain diagrams for similar wire strengths were assumed for these steels. The following stress-strain diagrams have been used in these cases.

1. Stress-strain curves for Types I, II, and III steels given in Billet's thesis (2).
2. Stress-strain curves for three typical high tensile strength wires reported by Revesz (3).

3. Stress-strain curves for six typical wires reported by H. J. Godfrey (4).

In general, the procedure for solving for the ultimate steel stress was as follows:

1. Known -- p, f'_c, f_{se}, k_2, ϵ_u, (stress-strain curve for steel either known or assumed).
2. Known or computed values of $k_1 k_3 f'_c$, p, ϵ_u, ϵ_{se}, and ϵ_{ce} were inserted into Eq. 2.
3. Using the stress-strain curve for the steel, a reasonable value of f_{su} corresponding to ϵ_{su} was assumed and ϵ_{su} subsitituted into Eq. 2.
4. Equation 2 is solved for f_{su}. If the value of f_{su} from Eq. 2 is reasonably close to the assumed value of f_{su}, f_{su} from Eq. 2 is taken as the stress in the steel at failure of the beam. If the assumed and computed values of f_{su} do not check closely, then new values of f_{su} and ϵ_{su} are assumed and the process repeated until the assumed and computed value check.

After the ultimate steel stress was computed, it was necessary to compute the depth to the neutral axis in order to determine the effective lever arm at failure and, in the case of I, T, and box sections, to determine whether the compression zone at failure was entirely within the flange. In only one beam test reported here was the computed depth of the compression zone at failure less than the depth of the flange.

If the depth to the neutral axis, computed on the assumption of a rectangular compression zone, is greater than the flange depth, then the strength of the beam is governed by the strength of the flange in compression. The beam cited in reference 7 failed in this manner. Assuming the compression zone to be rectangular, the computed steel stress at failure was about 200 ksi, and the neutral axis was computed to be 5.6 in. deep while the flange thickness was 4.0 in. Assuming only the flange to act in compression with an average stress of $k_1 k_3 f'_c$, the steel stress was computed to be 145 ksi, giving a ratio of measured to computed moment of 0.96. Thus, a different procedure must be used for stress and moment, computations if the neutral axis falls below the flange at failure.

For beams with rectangular compression zones at failure, the ultimate moment of the beam was computed by use of Eq. (4) and the computed values of f_{su} and k_u.

III. PRESENTATION OF TEST RESULTS AND COMPUTED VALUES

A comparison of measured and computed ultimate moments is presented in Tables 1 and 2 together with the data reported in the tests and computed and derived quantities. Explanations for each column in Tables 1 and 2 are as follows:

Column	Explanation
(1)	The numbers in this column refer to reference numbers in the bibliography.
(2)	The numbers in this column are the original beam designations appearing in the various references.
(3)	The general shape of the beam is designated by R, T, I, B, or C, which refer to the shapes sketched in Fig. 1.
(4), (5), and (6)	The dimensions in inches refer to the corresponding shapes in Column 3 and Fig. 1.
(7)	The steel ratio is given as the area of steel divided by the product of the overall flange width b and the effective depth d.
(8)	The compressive strength of the concrete as obtained from standard 6 by 12-in. test cylinders at the time of test of the beam are given. Where cube strengths were given in the references, the cylinder strength has been taken as the cube strength divided by 1.18.
(9)	The tensile strength of the reinforcing steel is given in kips per sq. in.

Column	Explanation
(10)	The initial prestress is given in ksi.
(11)	The measured ultimate moment is given in in.-kips.
(12)	The value of $k_1 k_3$ calculated from Eq. 5 is given.
(13)	The dimensionless parameter, $Q' = \frac{E_s p}{k_1 k_3 f'_c}$ is given.
(14)	The ratio of the depth to the neutral axis to the depth of the beam, k_u, computed from the expression $k_u = \frac{f_{su} p}{k_1 k_3 f'_c}$ is given in this column.
(15)	The computed steel stress at failure is given in ksi.
(16)	The computed lever arm ratio, $1 - k_2 k_u$ is given.
(17)	The computed ultimate moment is given in in.-kips.
(18)	The ratio of measured to computed ultimate moment is given.

It may be pointed out that, in general, the values in Columns 3-11 in Table 1 are all data actually reported in the descriptions of the tests; whereas all the data listed in Columns 12-18 of Table 2 are computed values. In spite of the fact that in some of the test reports only limited data were given and assumptions were made as to the stress-strain curve for the steel, the agreement between measured and computed moments is good.

Additional studies of the data are shown in Figs. 2 and 3. In Fig. 2, the ratios of the ultimate stress in the steel at failure in the beam to the tensile strength are plotted in Fig. 2(a) against p, in Fig. 2(b) against Q' and in Fig. 2(c) against $f'_s p/k_1 k_3 f'_c$. As may be expected, an increase in p generally decreases the ultimate steel stress. Since the prestress and stress-strain properties of the reinforcement vary greatly, no exact correlation should be expected in Fig. 2. However, there seems to be less scatter in the plots when the ratio of p to $k_1 k_3 f'_c$ is one of the variables. In Fig. 3, the lever arm coefficients, $(1 - k_2 k_u)$ are plotted against Q'. It is interesting to note that for values of Q' less than 30, the scatter of the points is very small.

IV. ANALYSIS OF TEST RESULTS

Tables 1 and 2 indicate a good correlation between measured and computed ultimate moments. The range of the ratio of the two moments is 0.92 - 1.52; however, if two beams for which explanations are made are excluded, the range is 0.92 - 1.08. This correlation is to some extent a confirmation of the analytical procedure for computing the moments. The comparatively small amount of scatter in Fig. 2 for low values of Q' or p is also a confirmation of the predictions of the analysis reported previously (1) (2), that for beams with low values of Q', the steel stress at failure of the beam is near the tensile strength.

In Fig. 2, the point marked 7 falls below the other values. This beam failed with the neutral axis below the top flange, and consequently the compression zone was smaller than would have been had the beam been rectangular. The strength of the beam was limited by the strength of the compression zone, leading to a reduced steel stress at failure of the beam.

Although the types of steel used in the beams tested varied to a large extent, it was thought to be desirable to investigate by means of the analytical procedure the effect of the properties of the steel stress-strain curves on the computed values of f_{su}/f'_s. Several steel stress-strain curves were selected from various sources and are plotted in Fig. 4. They were as follows:

Wire	Type of Wire and Source of Stress-Strain Curve
A	Hard drawn wire - 0.2 in. diameter-from W. O. Everling's discussion of Reference (4).

Wire	Type of Wire and Source of Stress-Strain Curve
B and C	Stress-relieved wire - 0.099 and 0.276 in. diameter - from Reference (1).
D	Stress-Relieved Wire - 0.199 in. diameter - from Reference (2).
E	Stressteel Bar-Tensile strength, elongation, and part of stress-strain curve given in "Stress-Steel Manual."
F	An idealized curve whose yield point, yield strength, and tensile strength are the same.
G	An idealized curve that extends in a straight line between prestress stress and strain and fracture stress and strain.

The idealized curves F and G were selected as the absolute upper and lower limits of possible stress-strain curves for steels with a tensile strength of 240,000 psi, an elongation at failure of 0.04, and a prestress of 120,000 psi. The upper limit is more nearly approached by steels now available than is the lower limit.

For each of the stress-strain curves shown in Fig. 4, values of ultimate steel stress, f_{su}, for several values of Q' were computed as described previously in this report for beams of rectangular section with an effective tensioning stress of one-half the tensile strength. The ratios of f_{su} to f'_s have been plotted in Fig. 5 versus Q' and in Fig. 6 versus $f'_s p/k_1 k_3 f'_c$. In both of these plots, the curves for the

various steels group relatively close together considering the differences in their stress-strain curves. The upper and lower limit curves are quite different indicating the maximum possible spread in ultimate steel stress for a steel with that particular tensile strength and elongation. In all cases, except for the lower limit curve, the computed ultimate steel stress for values of Q' below about 30 is between 90 and 100 percent of the tensile strength of the steel. While no known specific relationship exists between ultimate steel stress and Q' or $f'_s/k_1k_3f'_c$ for all types of steel, it seems likely that the comparisons with tests and the computations show that for the steels and beam shapes considered, the steel stress at failure of the beam will be from 90 to 100 percent of the tensile strength of the steel if Q' is less than 30 and if the tensioning stress is about one-half the tensile strength or greater. It is believed that the steels considered have stress-strain properties which may well cover most of the variation in properties in steels available at the present time.

V. SUMMARY AND CONCLUSIONS

A study of tests on prestressed concrete beams has been made with the object of comparing their failure moments with those computed by use of Equations 1-5. An excellent overall agreement between measured and computed moments was obtained. The results indicate that for a certain range of the ratio of p to f'_c as expressed by Q', the steel stress at failure of the beam may be taken without serious error as a certain percentage of the tensile strength of the steel reinforcement. Further analytical studies involving various types of steels having a variety of stress-strain properties confirmed the previous statement regarding the ratio of the ultimate stress in the steel at failure of the beam to the tensile strength of the steel. In only one case studied was the neutral axis at failure of the beam computed to be in the top flange of the beam, thus very little information was obtained relative to cases where the neutral axis at failure was below the top flange of the beam.

The major conclusions drawn from this study are as follows:

1. The analysis presented in the First Progress Report (1) and by Billet (2) may be used to predict the ultimate flexural strength of full size prestressed concrete beams if the neutral axis is in the top flange at failure of the beam. As a result of the good agreement between measured and computed moments, it may be concluded that the combination of empirical values for k_2, k_1k_3, and ϵ_u given by Billet (2) will lead to satisfactory predicted values of ultimate moment of bonded, prestressed concrete beams.

2. If Q' is less than about 30, and if the effective tensioning stress is about one-half the tensile strength or greater, f_{su} may be assumed to be about 0.90-0.95 f'_s. This conclusion of course must be limited to beams having steels with stress-strain properties within the limits covered by this study.

Emphasis should be given to the fact that this study covered only beams failing in flexure. If a beam fails in shear, bond, or as a result of anchorage failure, the failure moment will be less than the computed flexural ultimate.

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TABLE 1

MEASURED VALUES FOR BEAMS

Ref. No. (1)	Beam No. (2)	Beam Shape (3)	b in. (4)	t in. (5)	d in. (6)	p A_s/bd percent (7)	f'_c psi (8)	f'_s ksi (9)	f_{si} ksi (10)	M_{ult} Meas in.-kips (11)
5	1	T	15.75	7.875	28.75	0.162	7250	245	120	4350
5	2	T	15.75	7.875	28.75	0.162	6820	245	120	4900
5	3	T	15.75	7.875	28.75	0.162	6040	245	120	5180
5	4	T	15.75	7.875	28.75	0.162	8200	245	120	5310
5	5	T	15.75	7.875	28.75	0.162	6380	206	---	3730
5	6	T	15.75	7.875	28.75	0.162	7110	206	---	4450
3	IA	R	2.92	-----	4.60	0.776	6620	300	175	112
3	IB	R	2.91	-----	4.84	0.74	6650	300	175	124
3	IID	T	4.25	3.5	6.92	0.355	3400	300	173	167
3	IIL	T	21.5	1.5	7.46	0.066	5570	300	180	240
3	IIG	T	21.5	1.5	7.01	0.069	4700	300	110	222
3	IIF	T	21.5	1.5	7.74	0.063	1930	300	0	242
6		B	17.75		16.5	0.258	3000	224	153	3180
6		I	9.25		44.5	0.546	2700	146	78	11900
7		B	13.5	4.0	34.3	0.425	6800	206	113	8350
8		I	52		70.25	0.418	5500	247	131	230,000
9		I	7		13	0.62	3000	224	150	1090
10		C	13.5	7.0	30	0.495	3520	224	150	1880
11	P1	I	7.0	2.0	13.0	0.64	7200	224	150	1410
11	T_1	C	10.0		13.5	0.695	6000	224	---	2380
11	T_2	C	10.0		13.5	0.695	7200	224	---	2310
12	A	I	9.0	1.9	10.4	0.641	5090	156	94	676
12	B	I	9.0	1.9	10.4	0.641	6000	156	94	776
12	C	I	9.0	1.9	10.4	0.641	6780	156	94	818
12	E	B	9.25		19.4	1.00	5090	156		3870
13		I	18.0	5.0	23.0	0.35	5000	230	120	9300
13		I	30.0	5.0	37.0	0.25	5000	210	120	24,000
14	S-3	C	18.5	2.0	10.5	0.485	3650	224	150	186
14	S-4	C	18.5	2.0	10.5	0.485	3000	224	150	170
15	20-H	R	8		16.75	0.178	4600	269	0	1040
15	20-L	R	8		16.75	0.178	3380	269	0	990
16		I	9		10.5	0.605	4930	156		771
16	FJ105	I	2	1	3.65	0.71	5950	230	184	35.6

TABLE 2

COMPUTED VALUES FOR BEAMS

Ref. No.	Beam No.	k_1k_3	$Q' = E_s p/k_1k_3f'_c$	k_u	f_{su}	$1-k_2k_u$	M_{ult} meas. in.-kips	M_{ult} comp. in.-kips	meas./comp.	Remarks
(1)	(2)	(12)	(13)	(14)	(15)	(16)	(11)	(17)	(18)	
5	1	0.76	7.5	0.069	235	0.97	4350	4780	0.92	
5	2	0.77	7.4	0.074	240	0.97	4900	4800	1.02	
5	3	0.80	8.5	0.080	240	0.97	5180	4850	1.06	
5	4	0.73	7.0	0.066	245	0.97	5310	4900	1.08	
5	5	0.78	7.7	0.065	200	0.97	3730	4080	0.92	Beam listed as defective
5	6	0.76	7.6	0.062	206	0.97	4450	4200	1.06	
3	IA	0.78	45.0	0.373	250	0.84	112	102	0.96	
3	IB	0.78	42.8	0.393	275	0.84	124	116	1.07	
3	IID	0.96	32.7	0.300	275	0.87	167	173	0.96	
3	IIL	0.82	4.3	0.043	300	0.98	240	230	1.04	
3	IIG	0.86	4.4	0.051	300	0.98	222	215	1.03	
3	IIF	1.16	9.8	0.084	300	0.96	242	235	1.03	
6		1.00	27.5	0.195	200	0.92	3180	3100	1.02	
6		1.02	55.0	0.260	130	0.89	11,900	11,900	1.00	
7		0.77	24.3	0.081	145	0.97	8350	9140	0.96	Neutral axis below flange
8		0.82	25.0	0.222	240	0.91	230,000	234,000	0.98	
9		1.00	62.0	0.351	170	0.85	1090	1065	1.02	
10		0.95	44.7	0.331	224	0.86	1880	1240	1.52	N.A. below cast in place conc.
11	P1	0.76	35.0	0.252	216	0.89	1410	1460	0.96	
11	T_1	0.80	43.5	0.305	210	0.87	2380	2330	1.02	Partially prestressed
11	T_2	0.76	38.0	0.274	216	0.88	2310	2410	0.96	Partially prestressed
12	A	0.84	45.0	0.195	130	0.92	676	700	0.96	
12	B	0.84	38.0	0.210	140	0.91	776	795	0.98	
12	C	0.77	36.5	0.172	140	0.93	818	820	1.00	
12	E	0.84	70.0	0.305	120	0.88	3870	3970	0.97	
13		0.85	23.0	0.182	220	0.92	9300	6820	1.36	f'_c not given at time of test
13		0.85	16.5	0.125	210	0.95	24,000	20,600	1.16	f'_c not given at time of test
14	S-3	0.89	42.5	0.250	200	0.88	186	175	1.06	Partially prestressed
14	S-4	1.00	45.5	0.324	200	0.86	170	170	1.00	Partially prestressed
15	20-H	0.87	12.6	0.121	260	0.95	1040	1000	1.04	
15	20-L	0.96	15.4	0.143	260	0.94	990	980	1.01	
16		0.85	40.5	0.197	135	0.92	771	770	1.00	
16	FJ105	0.81	34.5	0.324	220	0.86	35.6	35.5	1.00	

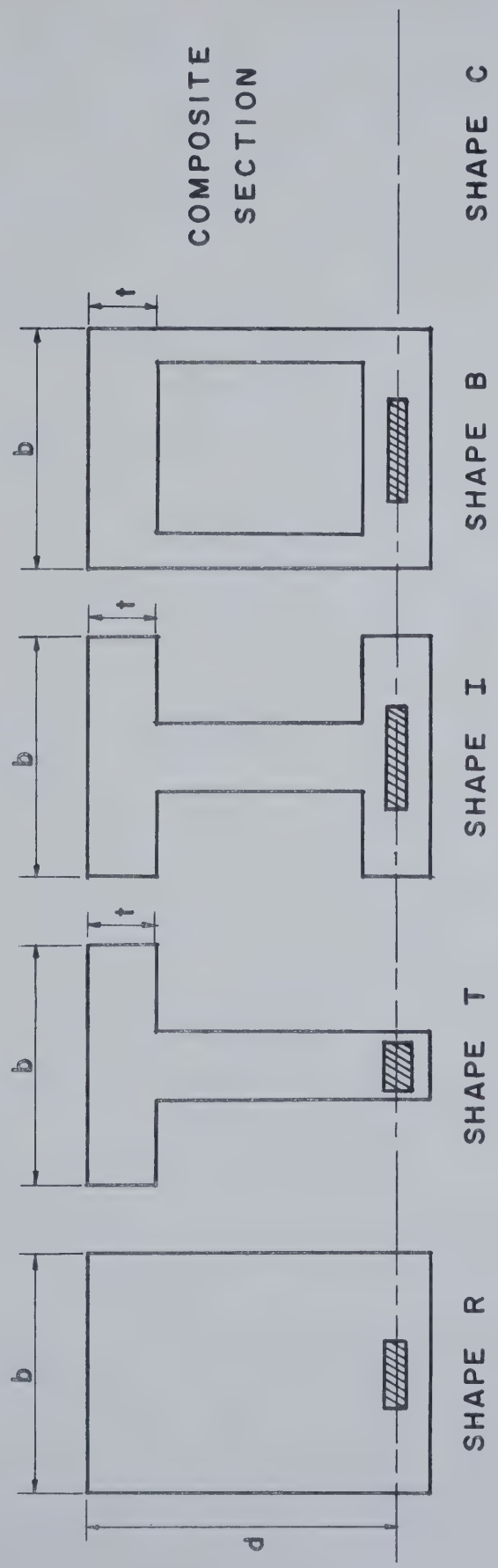

FIG. 1 GENERAL CROSS-SECTIONS OF BEAMS STUDIED

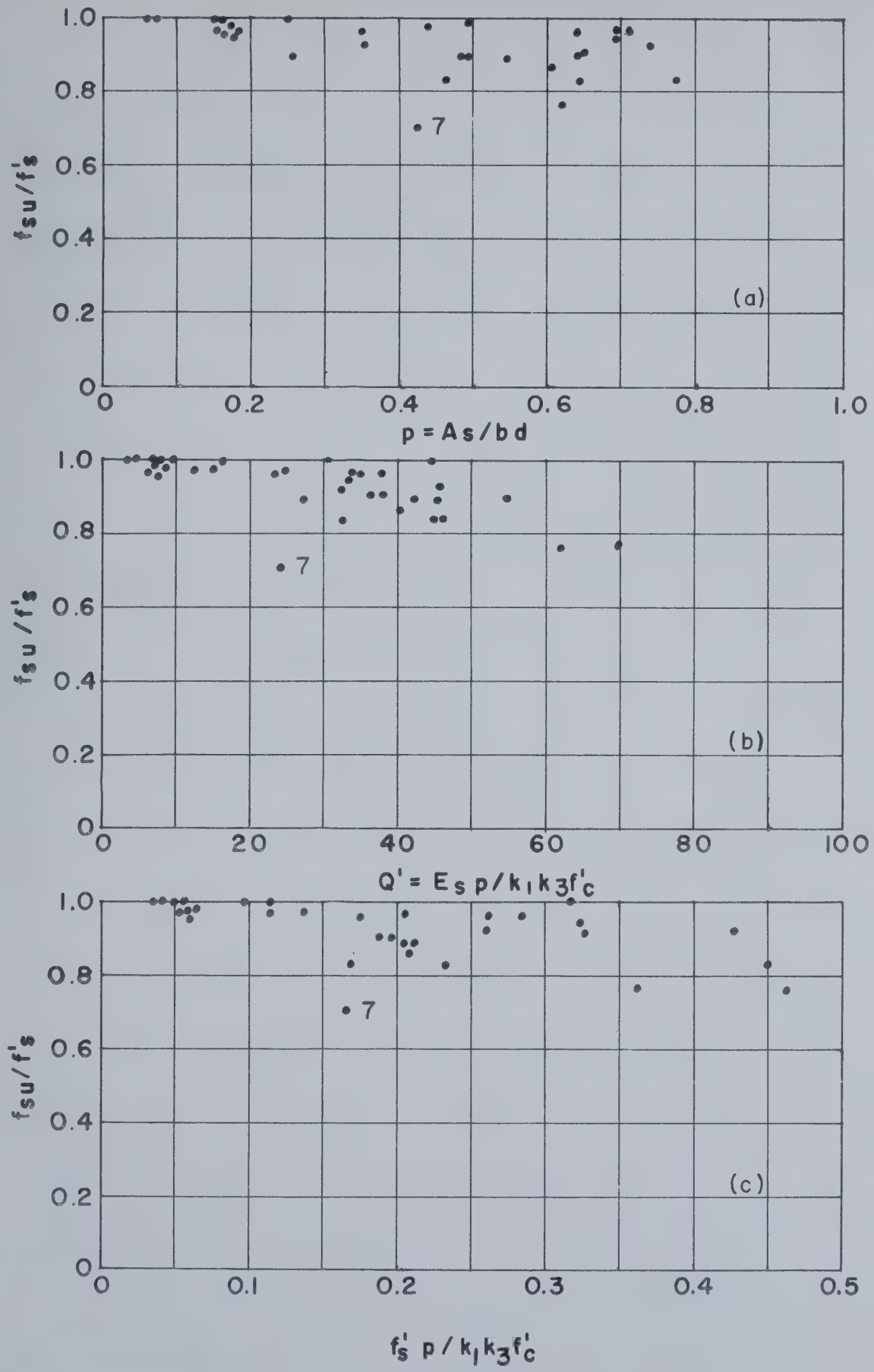

FIG. 2 VARIATION OF f_{su}/f'_s WITH p, Q', AND $f'_s p/k_1 k_3 f'_c$

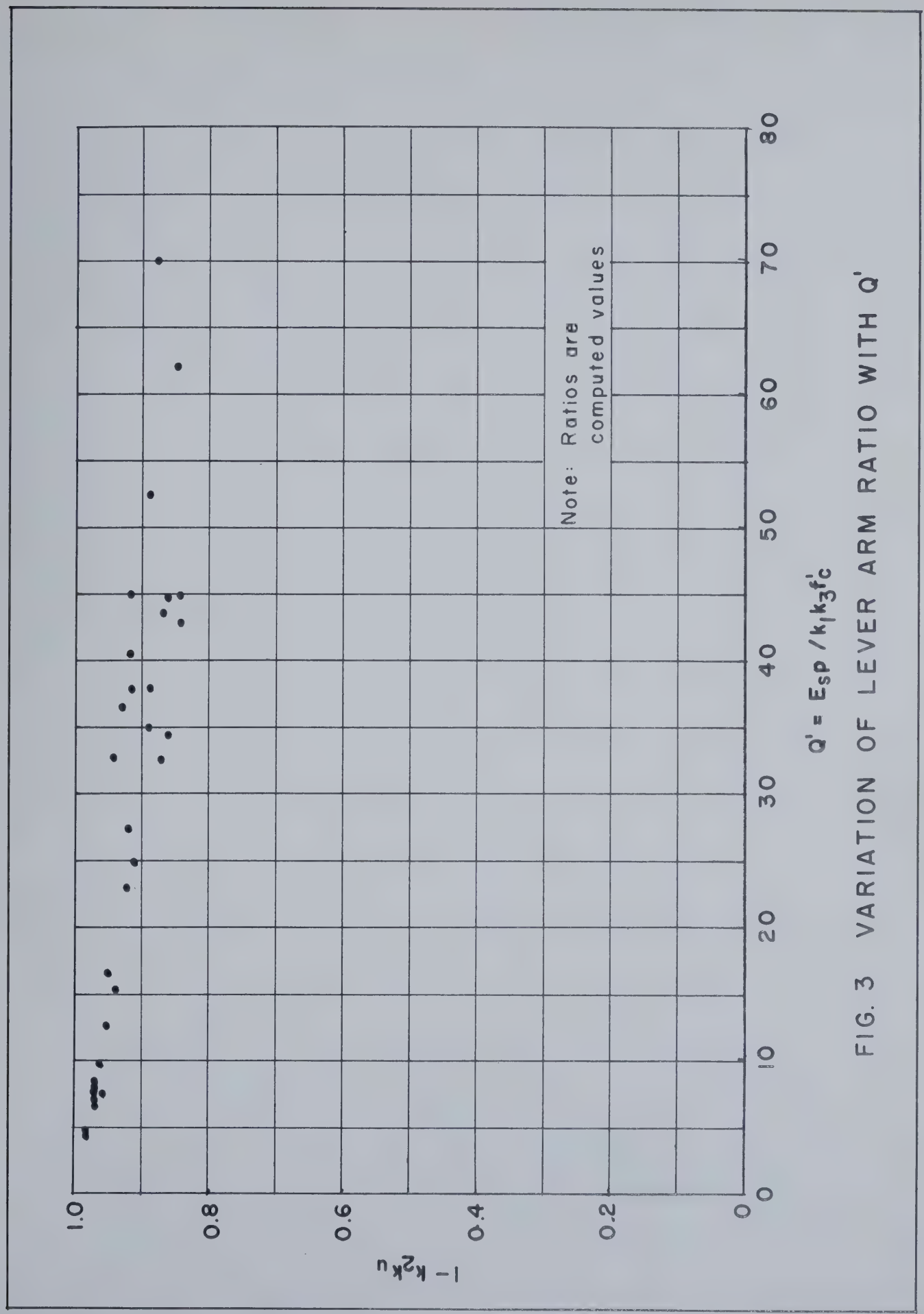

FIG. 3 VARIATION OF LEVER ARM RATIO WITH Q'

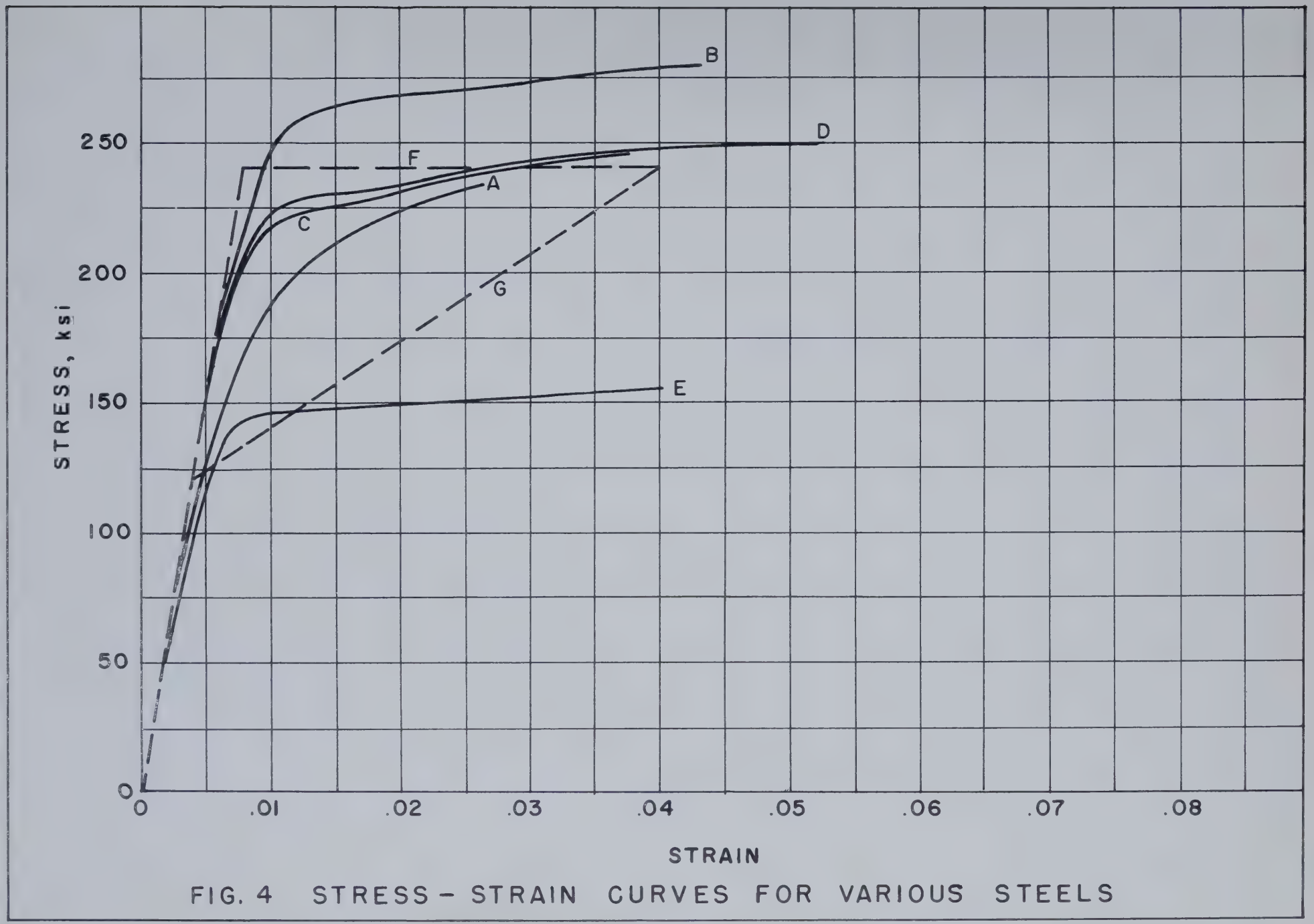

FIG. 4 STRESS – STRAIN CURVES FOR VARIOUS STEELS

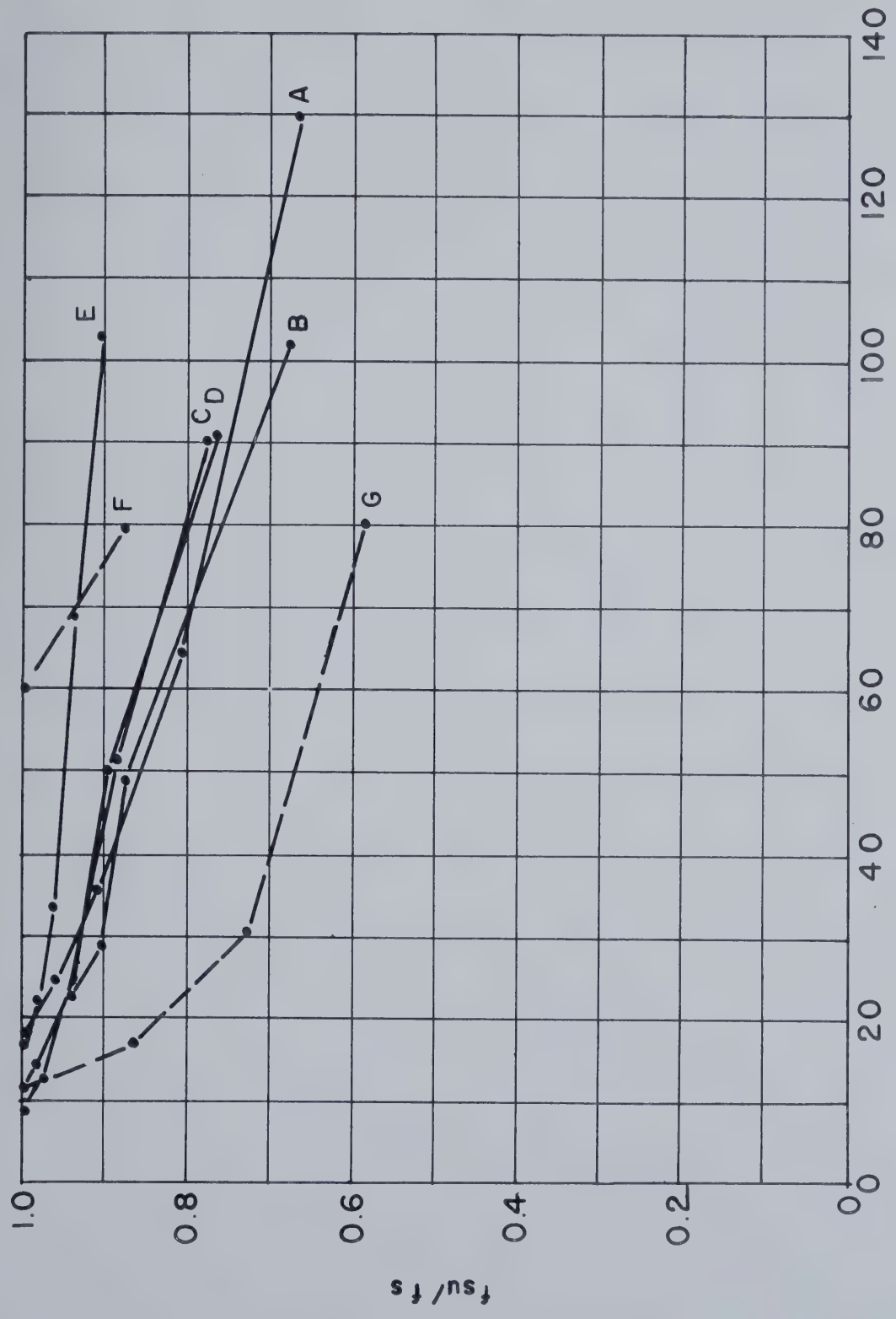

FIG. 5 RATIO OF ULTIMATE STRESS TO TENSILE STRENGTH VERSUS Q'

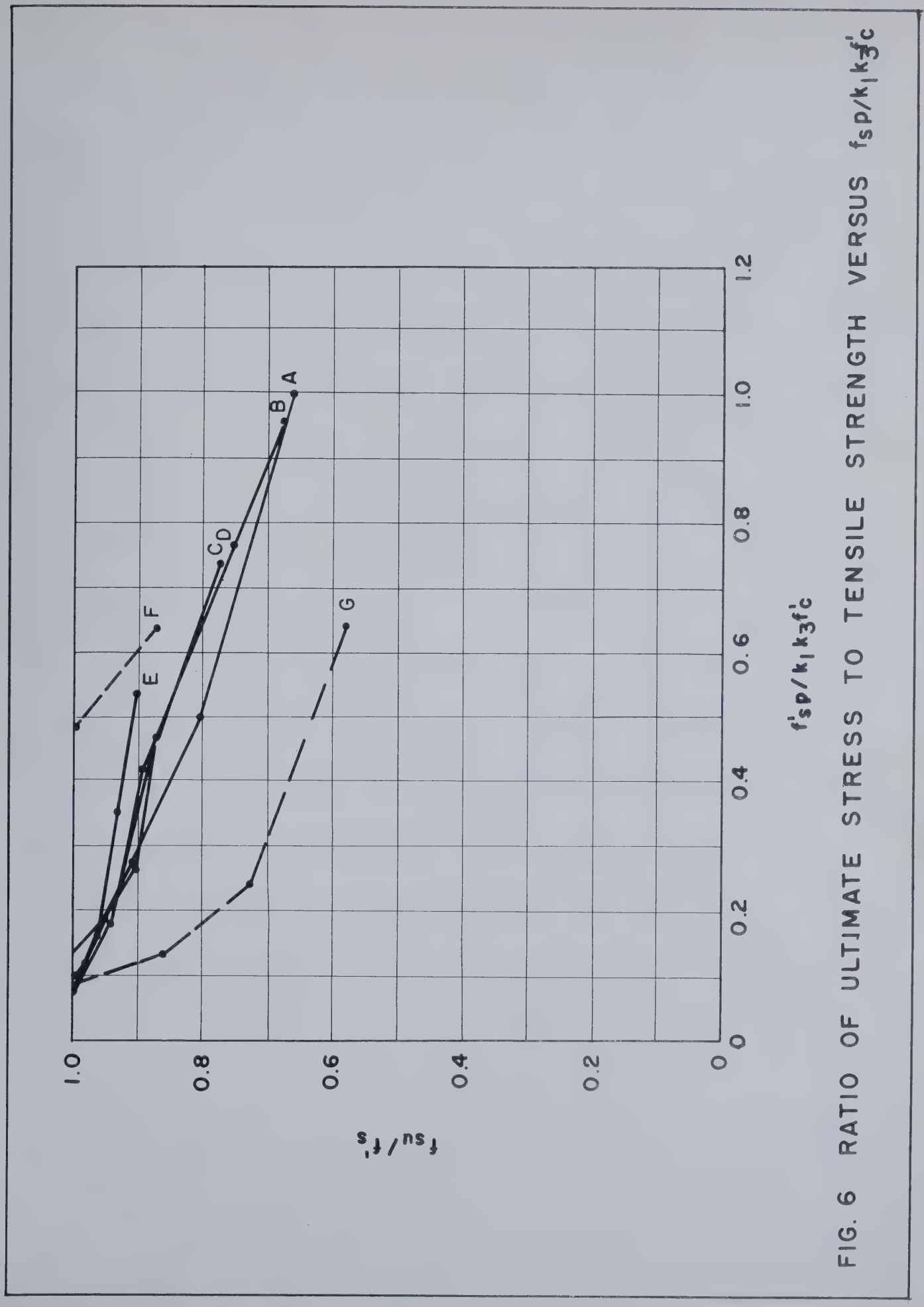

FIG. 6 RATIO OF ULTIMATE STRESS TO TENSILE STRENGTH VERSUS $f_{sp}/k_1k_3f'_c$

PART C

PROGRESS REPORT

on

BEAMS FAILING IN SHEAR

by

M. A. Sozen

C. BEAMS FAILING IN SHEAR

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C. TESTS OF BEAMS FAILING IN SHEAR

I. INTRODUCTION

1. Object and Scope

The advent of prestressing into the field of reinforced concrete construction has given rise, along with numerous possibilities of application, to an impressive number of questions. Quite in contrast to the conservatism exercised in the adoption of ordinary reinforced concrete, the profession has taken to prestressed concrete with unusual ardor and confidence. Consequently, even though there has been overwhelming progress in the applications of prestressed concrete, more than a few questions of research remain unanswered today. Among them lies the investigation of shear failures, an aspect which has appeared trivial to many designers. Yet, in view of the developing trend in design concepts of evaluating the strength of structural elements on basis of their ultimate strength, the very existence of shear failures makes it imperative that this phase of the prestressed concrete research be fully investigated.

In connection with this investigation of prestressed concrete E. M. Zwoyer has initiated a test series concerning the ultimate shear strength of simply supported rectangular prestressed concrete beams.* The results of these tests have been interpreted into analytical expressions with empirical factors giving the ultimate shear strength of such members.

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*"Shear Strength of Simply Supported Prestressed Concrete Beams," E. M. Zwoyer, Issued as a part of the Second Progress Report of the Investigation of Prestressed Concrete for Highway Bridges, June 1953.

The I-Beam is the most common type of beam used in prestressed construction. Since it appeared that changes in geometry of cross-section would modify the strain distribution through the depth of the beam, and since the empirical data for this distribution was very influential in the derivation of the expression for ultimate capacity of rectangular beams, the need for the series of tests on I-Beams reported herein was felt.

This report is a direct presentation of data from tests completed and general trends observed up to date. No effort has been made to correlate conclusively the test data to any particular analytical expression.

The present objectives of the investigation, subject to modification in the event of new trends in the test data, may be summarized as follows:

1. To verify the validity of the empirical expression for the ultimate shear strength of rectangular reinforced concrete beams at zero prestress.

2. To check the dependability of the proposed method, based on a prediction of the ultimate steel stress, of distinguishing between shear and flexure failures.

3. To determine the modes and characteristics of shear failures in simply supported prestressed concrete I-Beams without web reinforcement.

4. To evaluate the effects of the following variables on the ultimate shear strength of prestressed concrete I-Beams:

a. concrete strength

b. amount of longitudinal steel

c. ratio of shear span to effective depth of beam

d. prestress in the steel

e. thickness of the web

5. To investigate whether the shear strength of I-Beams can be predicted on the basis of an empirical relationship between the concrete and steel strains at failure, and, if so, to establish that relationship.

6. To form a basis for the planning of future tests.

At present tests have been completed on thirty-one beams, five of which were 6 by 12 in. 10 ft. rectangular beams. These were post-tensioned, end-anchored and grouted. The overall dimensions of the twenty-six I-Beams were 6 by 12 in. by 10 ft.-10 in. Nine of them had a nominal web thickness of 1 3/4 in., this dimension being 3 in. for the remaining seventeen. (See Figs. 1a and 1b). They were all pre-tensioned and later end-anchored to prevent any possible slip of the reinforcement. All of the test specimens were loaded at the third-points over a span of 9 ft.-0 in. except for three 3-in. web I-Beams which were subjected to a single central load over the same span.

2. Outline of Tests

The properties of the test specimens are tabulated in Table 1. No compression or web reinforcement was placed in the beams. The main variables in the thirty-one beams are discussed in the following paragraphs with the exception of changes in web thickness which have been mentioned in the preceding section.

The concrete strength was varied approximately from 1750 psi to 6000 psi. The minimum strength, which is too low to be of any practical consideration, had to be resourced to for the purpose of having a high ratio of percentage of longitudinal reinforcement to concrete strength without developing too large a prestressing force. No higher strengths than 6000 psi were strived for in view of the excessive shrinkage that would be engendered by the richness of the mix, as the size of the aggregate demands a high cement content to attain high strengths. Nonetheless, the range was ample to bring to light fully the influence of concrete strength on the ultimate shear capacity.

There could not be secured as desirable a range of variation in the amount of longitudinal steel as in concrete strength. The percentage of steel was restricted on the upper bound by the limited flange area and the necessity of maintaining the prestressing force at a low enough value so as not to severely crack the top flange, and on the lower bound by the requirement for shear failures of a minimum ratio of percentage of longitudinal steel to concrete strength. Maximum and minimum percentages of longitudinal steel were 0.606 and 0.181 respectively.

Twenty-eight specimens were tested with third point loading and three were loaded by a single load at the center of the span. Fig. 1a illustrates both loading arrangements. The latter arrangement was chosen with a view to investigate the effect of change in the shear span on shear strength.

All specimens, except S-2a which was cast to simulate S-2 of the previous series, were prestressed to one of the three selected nominal prestresses. Four rectangular and eight I-Beams had zero prestress, nine I-Beams had a nominal prestress of 60,000 psi and nine others were tensioned to a nominal prestress of 120,000 psi.

3. Definition of Terms

The reader is referred to Section 5 of E. Zwoyer's dissertation on the shear strength of simply supported prestressed concrete beams for all definitions and notations used in this report.

II. MATERIALS AND FABRICATION

4. Materials

(a) Cements: The cements used were Marquette Type I and Marquette Type III for the rectangular beams and I-Beams respectively.

(b) Aggregate: Wabash River gravels and Wabash River sand were used as aggregate. The maximum size of coarse aggregate was 1 in. for the rectangular specimens and 3/8 in. for the I-Beams. Aggregate sieve analyses are reported in Table 2.

These aggregates have as their origin Wisconsin Glacial outwash. The principal constituents of the gravel are limestone and dolomite with minor quantities of quartz, granite, gneiss, etc. The sand consists mainly of quartz. The absorption of both the coarse and the fine aggregate was about one percent by weight of surface dry aggregate.

(c) Concrete Mixes: Mixes were designed by the trial batch method and yielded concrete strengths ranging from 1750 psi to 6040 psi as measured by 6 x 12 control cylinders. (See Table 3). The mixes were proportioned on the basis of a slump of 3 in. Moisture samples were taken from the sand and gravel used in each mix and the reported water:cement ratios were based on their results.

(d) Reinforcing Wire: Four types of reinforcing wire, called Type I, Type II, Type VI, Type VII from two sources have been used in the current series.

The wire designated as Type II and used only for specimen V12 is a product of the John A. Roebling's Sons Corporation. It is an acid-steel wire stress relieved by heat treatment after drawing. Its stress strain characteristics are shown in Fig. 2b.

The other three types, all made from basic open hearth steel, were manufactured by the American Steel and Wire Company, a division of the United States Steel Corporation and are designated in general as Super-Tens wire.

The manufacture and analysis of Type I wire has been reported in Section 6e of E. Zwoyer's thesis. The stress-strain characteristics for it are given in Fig. 2a of this report.

The manufacture of the wires referred to as Types VI and VII comprised the following procedure (1) The ingots were hot-rolled into rods of suitable size. (2) The rods were subjected to a patenting process, i.e., they were quenched from an austenitizing temperature by immersion in hot lead. (3) The rods, then, were cleaned, lime coated and wet-drawn through wire-drawing soap to finished size, the percentage of reduction in area depending on the required ultimate tensile strength. (4) The wire was stress relieved for a final time by immersion into hot lead at a temperature of about 800° F. The time of exposure to lead depended on the size of the wire and ranged from 5 to 15 seconds.

Type VI wire was recieved in two five-foot diameter coils of about 114 lbs. and 98 lbs. American Steel and Wire Company assigned it the following analysis: Carbon 0.82 percent; Manganese 0.73 percent; Phosphorus 0.010 percent; Sulphur 0.034 percent; Silicon 0.20 percent. The average diameter of the wire as measured from tensile test samples was 0.1925 in. Its stress-strain characteristics are shown in Fig. 2c.

Type VII wire was recieved in a similar five-foot diameter coil weighing about 198 lbs. and was reported to have the following analysis: Carbon 0.86 percent; Manganese 0.87 percent; Phosphorus 0.010 percent; Sulphur 0.025 percent; Silicon 0.18 percent. The average measured diameter of the wire was 0.1945 in. As may be observed in Fig. 2d this type of wire had a considerably higher ultimate strength than the others.

All steel samples cut from different portions of each coil were tested in a 120,000 lb. capacity Baldwin Southwark Tate-Emery hydraulic testing machine. The strains were measured with an 8-in. extensometer and recorded with an automatic recording device.

In order to improve their bond to the concrete the wires were first wiped with a rag dipped in a weak solution of hydrochloric acid and stored in the moist room for several days. Insulating tape was wrapped around the wire at locations where electric strain gages were intended to be placed. On being removed from the moist room the rusted wires were cleaned with a wire brush to remove all loose rust.

5. Description of the Specimens

(a) Rectangular Beams: The rectangular beams were identical to those of the previous series, and the reader is referred to Section 7 of Zwoyer's thesis for their description and fabrication.

(b) I-Beams: The I-Beams were all pre-tensioned. The wires were released at time of removal of the specimen from the form. When the beam was placed in the machine and made ready for testing the reinforcement was anchored using end plates as a precaution against bond failures, a measure which proved unnecessary. Only longitudinal reinforcement, which extended in a straight line from one end of the beam to the other, was used.

Two types of I-Beams were cast, nine with 1 3/4-in. webs and seventeen with 3-in. webs. The webs were formed by metal and wood inserts placed in the forms used for the rectangular beams. All I-Beams had a total length of 10 ft.-10 in., a depth of 12 in. and a flange width of 6 in. The nominal dimensions for both types of I-Beams are shown in Figs. 1a and 1b.

To prevent splitting at the ends, the beams were also provided with four clamp-on stirrups placed immediately on both sides of the support.

6. Fabrication

(a) Prestressing:

(i) End Details of Wires. Threaded connections were used to grip the wire in the tensioning process. Specially heat-treated,

24-threads to the inch chasers in an automatic threading machine were utilized to cut the threads on the end three inches of the wires. Despite the heat treatment, the chasers were worn dull after threading fifteen to eighteen wires and required resharpening. The threads on the wires were cut to provide a medium fit with the threads in the nuts, giving rise to a thread slightly larger than a No. 10 with a basic major diameter of 0.190 in.

The nuts were specially manufactured in the laboratory machine shope. They were sub-drilled with a No. 16 tap drill and tapped with a standard No. 12, 24-threads to the inch tap. This provided a full No. 12 thread in the nuts. Nuts with a No. 10 thread required too much material to be cut from the wires to be practical. The thread cut on the wires to fit the No. 12 thread in the nuts was sufficient to develop at least 160,000 psi in the wires for several days and was considered to be the most suitable.

The nuts were made from "Buster" alloy punch and chisel steel of the following analysis range; Carbon 0.56-0.60 percent; Chromium 1.10-1.30 percent; Tungsten 2.00-2.30 percent; Vanadium 0.20-0.30 percent. The hardening process involved the following steps: (1) Packing in charcoal in a closed steel box. (2) Heating for 20 minutes at 1200° F. (3) Heating for 45-60 minutes at 1650° F. (4) Oil quenching to slightly above room temperature. (5) Tempering at 1000° F. for 30 minutes. (6) Removal from the furnace and air cooling.

Only hexagonal nuts 5/8 in. in length, shown in Fig. 6 of E. Zwoyer's thesis, were made use of as they could be applied and removed with ease.

(ii) Tensioning Apparatus. For a description of the tensioning apparatus refer to E. Zwoyer's thesis.

(iii) Measurement of the Tensioning Force. For a description of the measurement of the tensioning force refer to E. Zwoyer's thesis.

(iv) Tensioning Procedure. For a description of the tensioning procedure refer to E. Zwoyer's thesis.

(b) Mixing and Casting: The ingredients, in their prescribed proportions, were placed in the hopper of a 6-cu. ft. non-tilting drum mixer, the coarse aggregate first, then sand and finally cement. The hopper was emptied into the rotating drum and water was added a few seconds later. The time of mix was on the average 2 1/2 minutes during which time the mix was checked at intervals and one or two pounds of water added if it appeared dry. It took two 4-cu. ft. batches to pour one test specimen and the pertaining control cylinders and beams. Despite the use of one cu. ft. of priming mix prior to the main operation, the two batches were very seldom identical in strength, the second batch always having a tendency to be of lower consistency. In view of this difficulty care was taken to place the first batch in the bottom and the second in the top half of the beam to ensure as much informity of material in the compression flange as possible. The freshly poured concrete was adequately vibrated with a high-

frequency internal vibrator. At this stage two hooks were embedded in the beam at the quarter-points to facilitate handling.

For the first few beams the bulging out of the form was checked by a bolt going through the top flange at the center. It was encased in a steel sleeve spacer which was permanently left in the beam. However, as it was later noticed that this sleeve always initiated a crack before any load was put on the beam, two clamps were used externally at the third points for the same purpose.

Four 6 by 12-in. control cylinders and one 6 x 6 by 24 in. control beam was poured from the first batch and eight control cylinders and one control beam from the second batch. The top of the specimen was troweled smooth and the cylinders capped with a paste of neat cement four or five hours after the pouring operation. Four of the cylinders cast from the second batch were tested within the first six days in a 300,000 lb. hydraulic Riehle testing machine to estimate the time of release of the tensioned wires. The remaining cylinders and beams were tested on the day of the test to determine the concrete strength and the modulus of rupture respectively. The specimen was left in the form from three to five days depending on the concrete strength and the prestressing force. The sides of the form were pulled away from the beam within 24 hours of pouring to prevent the concrete from sticking to the inserts, for although the inside surface of the form was completely oiled, especially the high strength concretes exhibited a marked affinity to adhere to the wooden inserts placed in the form to shape the web.

The I-Beams were not subjected to any form of curing and were stored in laboratory air until time of test.

7. Electric Strain Gages

(a) Gages on the Reinforcing Wire: About half of the wires in each specimen had A7 SR-4 electric strain gages on them. The gages had a nominal length of 1/4" and a minimum trim width of 3/16".

The surface of the reinforcing wire, which had been kept free of rust by insulating tape, was smoothened out with emery cloth and cleaned thoroughly with acetone. The gage was attached to the wire with Duco cement. Drying was accelerated by the use of heat lamps. First insulating tape, and then cloth tape were wrapped around the gage and the uninsulated part of the leads. The cloth tape was impregnated by an application of Petrolastic, an asphaltic compound produced by Standard Oil of California, in the molten state. Care was exercised not to overheat this compound as bubbles, then evolved, would destroy the impermeability. The lead wires, designated Type FL No. 18 AWG Solid, were carried at the level of the steel to the end of the beam and the brought up and out of the top flange.

(b) Gages on the Concrete Surface: One-inch long A-11 SR-4 electric strain gages were used to measure the strains on the top surface of the beam. A portable grinder was utilized to polish the spots where the gages were to be applied. These locations were later cleaned with acetone and the gages attached with Duco cement. To secure full contact between the gage and the concrete surface, due

care was observed in their application and small steel weights were placed on the gages for a period of one hour. Since these gages were on somewhat moist concrete, no effort was made to waterproof them.

8. Strain and Deflection Measurements

Strains were measured at each load increment in the longitudinal reinforcement and on the top surface of the concrete by electric strain gages mentioned in Section 7.

Dynamometers attached to the protruding ends of the reinforcement were also read every other load increment to detect bond distress in the reinforcement.

Deflections were measured at the center of the span and at the third points with 0.001-in. dial indicators.

The cracks were marked at each increment of loading and the number of the increment at which the crack observed was inscribed on the beam right beside the crack mark. Photographs were taken at different stages of the test to be kept as a permanent record of the development of the crack pattern.

9. Testing Machines and Loading Rig

All specimens were tested in a 100,000-lb. Olsen screw-type testing machine. The load was measured with a 50,000 lb. elastic-ring dynamometer. The dynamometer was equipped with a dial indicator that was calibrated at 110.8 lbs. per division; it was sensitive to about one-tenth of a division. In order to provide access to the

beams during test, they were not placed in the middle of the machine, but to one side. The loading apparatus is shown in Figs. 12 and 13 of E. Zwoyer's thesis. For midspan loading the loading beam was removed and the load block was placed directly under the dynamometer. The load blocks are steel plates 6 in. by 2 in. by 8 in. with the longer side oriented in the direction of the span. Two pieces of leather were inserted in between the beam and each load block so as to leave a surface about two inches wide along the center line free of contact with the steel. This measure was taken for the correct functioning of the electric strain gage to be applied at that location. The leather pieces were four inches long in the direction of the span and 1/4 inch thick. The ball and roller combination provided for the movements that occurred at the bearings during the later stages of the tests when deflections were high.

10. Test Procedure

Normally, failure load was reached in about ten increments. Two or more center line deflections were recorded between successive increments. Loading was interrupted and all readings referred to in Section 8 were taken after each increment. Until the observation of the first crack, load increments were equal; after then their magnitude depended on the development of the crack pattern.

A certain amount of drop-off in the load occurred due to the increase in deflection while readings were being recorded. Therefore, the load and mid-span deflection were measured immediately after the interruption and before the resumption of loading.

Loading was continued in every case to complete failure. Each test took from three to four hours.

III. PRESENTATION AND DISCUSSION OF TEST RESULTS

11. Rectangular Beams

Beam S-2, tested by E. Zwoyer and reported in his thesis, failed in flexure although its value of Q' indicated that the steel stress would not reach the 0.2 percent offset stress, a criterion which was assumed to be the line of demarcation between shear and flexure failures. For a prestress of 50,000 psi this criterion implies that all simply supported beams of rectangular cross section with Type I reinforcement having a Q' of approximately 12.0 or more will fail in shear. The Q' for S-2 was 21.4 well up in the domain of shear failures. Two explanations suggest themselves for this inconsistency. The first is the possibility of a discontinuity in the line of demarcation between shear and flexure failures at effective prestresses of around 50,000 psi. The variables affecting this limit and their apparent mathematical relationships to it do not justify such an approach. The second, a purely irrational one, is attributing the cause of this phenomenon to the element of chance, i.e. saying that the shear failure was prevented by a beneficial arrangement of the initial cracks which may have relieved the web of distress due to diagonal tension. The second explanation seemed more plausible; however, Beam S-2a was cast to the specifications of S-2 and tested. It failed in shear at an ultimate moment quite consistent with the rest of the beams in the series. The results are reported in Tables 6 and 7.

The properties and test results of Beams V1 and V4 are listed in Tables 1, 6, and 7. Of these V2 and V4 failed in flexure and shear respectively as indicated by their values of Q'. Beams V1 and V3 failed in flexure although theory predicted shear failures for both.

It has been propounded that the ultimate steel stress a beam can develop can be used to distinguish between the probable modes of failure. A knowledge of $K\epsilon_u$, the effective prestress, and the properties of the beam will allow the computation of the ultimate steel stress. Figure 19a in E. Zwoyer's thesis shows the relation between the ultimate steel stress and the ratio $p/k_1k_3f'_c$ for Type I reinforcing wire. From this plot it is immediately observed that for an effective prestress of zero there is a palpable variation in the ultimate steel stress for a very small change in $p/k_1k_3f'_c$, especially for values of the latter below 0.5×10^{-6}. For Beam V1 $p/k_1k_3f'_c$ is 0.307×10^{-6} corresponding to an ultimate steel stress of about 190,000 psi which is below the 0.2 percent offset stress. However the value 0.307×10^{-6} is computed using a statistical relationship between f'_c and k_1k_3 which, as Fig. 28 of the Zwoyer Thesis shows, is subject to scatter. Therefore $p/k_1k_3f'_c$ may be as much as 25 percent off considering the case when the error introduced in the determination of the cylinder strength is additive to that in computing k_1k_3. Although the effect on the ultimate steel stress of this variation may not be greatly discernible in beams with moderate or high prestresses where f_{su} is a slower changing function of $p/k_1k_3f'_c$, it asserts itself vehemently in the determination of the ultimate steel stress for beams with no prestress. Therefore

it appears that the failure in flexure of V1 and also of V3 are not indications of the non-feasibility of the assumption on which the line of demarcation of shear and flexure failures is based. These failures do point out, however, that if the computed Q' is close to the limiting value, there can be made no infallible prediction of the mode of failure on the basis of the ultimate steel stress.

12. Behavior of I-Beams During Test to Failure

The general behavior of the test specimens V5 to V30 during test to failure may be divided into three distinct phases.

At first the beam behaves elastically, that is the observed increments of deflection vary linearly with the applied load. The extent of this stage depends primarily on the percentage of steel and the effective prestress, and secondarily on the concrete strength of which both the moduli of elasticity and rupture are assumed to be functions. Consequently beams having the same depth-span ratio and the same prestressing force have their first tensile crack practically at the same deflection. This has been observed to be true, the cracking deflections being approximately 0.02, 0.07 and 0.12 in. corresponding to nominal effective prestresses of zero, 60 and 120 ksi respectively, as the variation in p was not extreme.

The appearance of the first vertical crack, usually at mid-span naturally destroys the linearity of the load deflection relationship and marks the beginning of the second phase. The rate of increase of load starts to lag the rate of increase of deflection. Before the

center crack reaches the top flange, two more vertical cracks almost simultaneously open up on the bottom flange at or very close to the third points. None of these tensile cracks penetrates into the top flange. They stop in the web at about a height of two thirds of the depth of the beam. As the load increases more tensile cracks appear at locations consistent with the shape of the moment diagram and probably with the zones of weakness occurring along the length of the specimen. The main characteristic of this phase distinguishing it from the first and third phases is that the deflection is not a linear function of the load and also that all of the cracks are vertical. (Fig. 3a).

The transformation from the second phase to the third may take place in two different ways. In some cases a tensile crack, located at a distance, approximately equal to the effective depth of the beam from the point of application of the load towards the support, may initiate a diagonal crack which gradually propagates towards the load block. For all of the specimens with zero prestress and for some with a nominal prestress of 60,000 psi this crack leads to final failure. In some of the latter type of specimens failure is caused by a flatter crack which opens up suddenly at a later stage of loading. (Fig. 3b)

In the case of beams with greater prestressing forces a diagonal crack inclined at about 30° to the horizontal appears directly and violently at a load very close to the ultimate load.

In all instances the diagonal cracks penetrate into the top flange curtailing drastically the area of the concrete in compression. The strain gages on the top flange at the two outside fifths of the span indicate tension. Sometimes a tension crack may develop through the depth of the top flange in that region. As deflections increase the diagonal cracks, which have already transgressed into the top flange, stop their vertical movement, and finally distress is observed in the thus restricted compression zone adjacent to or under one of the load blocks; whereafter, the beam fails, the violence of the failure depending on the prestressing force and the concrete strength. Typical failures are illustrated in Fig. 3c.

13. Load-Deflection Relationships

The load-deflection relationships are shown graphically in Figs. 4a, 4b, 4c and 4d. Figure 4a compares the effect of prestress on the load-deflection curves for V17, V26 and V22 for the zero, 60,000 and 120,000 psi prestress classes respectively. Besides raising the ultimate capacity of the beam, increase in prestress prolongs the 'elastic' range and makes the transition from the linear to the non-linear portion of the curve less abrupt.

The loads at which the first flexural and diagonal cracks were observed have also been indicated in the same figure. Assuming concrete to have a constant ultimate tensile strain, it follows that the increase with prestress of the flexural and diagonal cracking deflections and consequently of the corresponding loads is the direct effect of the prestressing force.

Figures 4b, 4c and 4d compare the load-deflection relationships of beams with the same nominal prestress with respect to the concrete strength, the percentage of tension reinforcement, and thickness of web.

There does not appear to be any consistent difference of shape or range between the load-deflection relationships exhibited by I-Beams with 3" and 1 3/4" webs having all other variables the same. The manifest weakness of specimen V5 in the 60,000 psi prestress series may be justifiably attributed to manufacturing defects rather than to factors inherent in its geometrical properties. The prestressing force for V5 was 12.6 kips while the concrete strength was a mere 2060 psi.

The stiffness of the beams in general seems to depend primarily on the concrete strength in the 'elastic' range and on the percentage of tension reinforcement in the 'inelastic' range both of which indications are consistent with analytical formulae describing the behavior of reinforced concrete beams. Naturally the ultimate strength increases with increase in either or both of these variables.

14. Comparison of Measured and Computed Cracking Loads

Table 4 compares the loads at which the flexural cracks have first been observed with computed cracking loads. The latter have been calculated using the ordinary flexure formula along with the principle of superposition of stresses and are the loads which theoretically cause a tensile stress in the bottom flange equal to the modulus of rupture.

The moduli of rupture have been determined, by the use of the ordinary flexure formula, from the results of third-point loading tests on control beams over a span of 1'-6". Of the results of the two control beams, those pertaining to the first batches have been used in the above calculations. The control beams poured from the second batches were merely used to check the results of the first beam and also to obtain more data on the relation of modulus of rupture to the concrete strength. Figure 5 compares the modulus of rupture with concrete strength.

15. Distribution of Concrete Strain on Top Surface of Beam

Typical distributions of average concrete strains as indicated by A-7 SR4 electric strain gages, are illustrated in Fig. 6. They unanimously indicate the concentration of the strain at the point of application of the loads. The ultimate concrete strains are shown plotted versus the concrete strength in Fig. 7.

16. Nominal Shear Stresses and Principal Tension Stresses

Present design practice has endorsed the use of nominal shear stress and principal tension stress as a measure of the susceptibility to shear failure of reinforced concrete and prestressed concrete members respectively. A discussion of their validity and relative merits is beyond the scope of this report. However, it is befitting to consider here their immediate bearing on beams tested in the current series of shear tests. Table 5 shows the computed values of nominal shear and principal tension stresses for the specimens. The former are based

on the celebrated formula $v = \frac{V}{bjd}$ with V corresponding to the ultimate load; the principal tension stresses are obtained from Mohr's Circles drawn for working loads assuming to be the lowest loads that introduce tension into the bottom flange. Principal tension stresses were calculated only for the beams with 120,000 psi nominal prestress as the working load for the lower prestresses based on the adopted assumption turns out to be too low to have any meaning.

The tabulated values of the nominal shear stress suggest that the shear strength may be a direct function of the concrete strength. Obviously this implication is merely a reflection of the moment-shear strength relationship as the shear span for all but three of the specimens was the same, and the variation in the percentage of longitudinal steel was not great. Results of I-Beams V28 and V30, which were tested with a single load at the mid-span, refute this trend. Comparing the nominal shear stress of specimen V30 to that of specimen V21, which has virtually the same concrete strength and prestress, it is seen that the two quantities are not compatible. Moreover, the difference in the amount of reinforcement in the two beams is not high enough to furnish any justification of ascribing this disparity to the effect of percentage of longitudinal steel. Attempts at using the web thickness in computing the shear stress have led to even greater scatter.

Several authorities on prestressed concrete design have advocated the use of web reinforcement only when the computed principal tension stress in a member exceeds the tensile strength of the concrete.

If the latter quantity is assumed to be about 8 percent of the compressive strength, Table V shows that web reinforcement is not required in any of the beams, yet all have failed in shear.

If the beam is to be fully prestressed for all combinations of loading, then, the insignificance of the computed principal tension stress will assure safety against a shear failure. Yet, as tests have indicated, this provision appears superfluous since no diagonal tension cracks have been observed to precede the flexural cracks; and, once the beam has cracked the computation of these stresses is unjustified. Attention should also be drawn to the fact that, for symmetrical loading at least, the computed principal stresses will have the same value for different shear spans while tests definitely indicate that the capacity of the beam, in terms of the load, will change with position of the load.

17. Significance of $K\epsilon_u$

E. Zwoyer has predicted the ultimate capacity in shear of rectangular beams successfully with the use of an empirical factor relating the ultimate concrete strain to the ultimate steel strain. This factor, for the rectangular beams at least, appeared to be a function of the effective prestress only, therefore, an expression in terms of the effective prestress was derived empirically for $K\epsilon_u$, where ϵ_u, the ultimate concrete strain, was assumed to be a constant. Conceivably, there exists an at least qualitatively similar expression for the I-Beams, $K\epsilon_u$ also being expected to vary with the geometry of the cross-section. Before attempting to derive this expression from

the test data it is worthwhile to investigate the sensitivity of the ultimate capacity in shear to variations in $K\epsilon_u$. This can be accomplished by plotting $\frac{M_{ult}}{k_1 k_3 f'_c\, bd^2}$ versus $K\epsilon_u$ for a representative value of $\frac{p}{k_1 k_3 f'_c}$. Figure 9 shows the computed values of $\frac{M_{ult}}{k_1 k_3 f'_c\, bd^2}$ for $\frac{p}{k_1 k_3 f'_c} = 10^{-6}$ against $K\epsilon_u$ at different effective prestresses. As indicated by the same figure $\frac{M_{ult}}{k_1 k_3 f'_c\, bd^2}$ increases very slowly with increase in $K\epsilon_u$, this rate decreasing with increasing prestress. From Eq. 7 in section 19 of the Zwoyer Thesis it is apparent that the ultimate steel stress is an inverse function of $\frac{p}{k_1 k_3 f'_c}$, therefore, for higher values of $\frac{p}{k_1 k_3 f'_c}$ the sensitivity of the ultimate capacity in shear to $K\epsilon_u$ is even less; lower values of $\frac{p}{k_1 k_3 f'_c}$ do not demand consideration as they lead to ultimate steel stresses above the 0.2 percent offset stress, indicating flexure failures, at all but the lowest effective prestresses. Hence the ultimate moment capacity is quite insensitive to variations in $K\epsilon_u$ for values of Q in the domain of shear failures. Conversely $K\epsilon_u$ is very sensitive to variations in the ultimate moment within the same range.

It follows that a slight experimental error in determining $k_1 k_3$ is considerably magnified in the value of $K\epsilon_u$ calculated from test data. Figure 10 is a plot of measured $K\epsilon_u$ versus effective prestress. It is justifiable to attribute a major portion of the scatter to the error inherent in the mathematical relation of this empirical constant to the test data.

18. Significance of k_1k_3

A brief study of Eq. (12) in Section 25 of E. Zwoyer's thesis shows that the most salient source of experimental error in the calculation of $K\epsilon_u$ is the measured value of k_u, or more basically $k_1k_3f'_c$, as all the other quantities are either insignificantly small or can be measured reliably. Values of $k_1k_3f'_c$ computed from measured values of k_u are plotted versus f'_c in Fig. 8. The wide scatter in the plotted points indicates the futility of attempting to obtain a statistical relationship between k_1k_3 and f'_c as the general pattern of the data will lend itself to almost any interpretation with equal success. Nonetheless it can be plausibly inferred from previous experience that $k_1k_3f'_c$ should be a particular, continuous and single valued function of f'_c; therefore, in lieu of using measured values of k_u to compute k_1k_3 directly, it may also be expedient to try to arrive at it by working back from the measured ultimate moment, a relevant value of $K\epsilon_u$ having been judiciously selected. Values of k_u predicted from equilibrium conditions by using values of $k_1k_3f'_c$ derived in this way are naturally not to be expected to be consistent with experimental data on k_u. On the other hand, the ultimate capacity for shear failure of the I-Beams may be predicted with success.

Another alternative is to assume k_1k_3 to be equal to unity and regard $K\epsilon_u$ as a function of the concrete strength whereby the ultimate capacity of an I-Beam failing in shear can be closely anticipated with the help of an empirical expression for $K\epsilon_u$ in terms

of both the effective prestress and the concrete strength, though there would be no direct way of foretelling the depth of the compression zone in the beam. However, the importance of this last quantity is debatable. It assumes primary interest only in its function as a direct means of determining the ultimate steel stress whence distinction be made between shear and flexure failures. Once the ultimate moment a beam is capable of carrying is known, the ultimate steel stress can be forthwith calculated to indicate the mode of failure without the necessity of precise knowledge of k_u as long as the beam under consideration is not very heavily reinforced in tension.

Figures 11a, 11b, 11c show the ultimate capacities of the test specimens plotted versus Q. For the sake of a qualitative comparison, empirical relations derived by Billet and Zwoyer for flexure and shear failures respectively have been superposed on the figures, the ultimate capacities having been calculated for a $k_1 k_3$ of unity and for Type I reinforcement which, for shear, is typical of all the wire types used. The experimental results, at least at effective prestresses of zero and 60,000 psi where fabrication conditions allowed a wider range of Q, exhibit a definite trend and appear to lie on a curve similar to Zwoyer's. There does not seem to be a marked difference between the ultimate capacities of I-Beams with different web thicknesses, a sign not too propitious for an attempt to relate $K\epsilon_u$ for rectangular beams to the same factor for I-Beams on the basis of web thickness. It should also be pointed

out that the quantitative relation of the experimental data to the theoretical curves on Figs. 11a, 11b, 11c is not of any significance as both of the theories are based on varying values of $k_1 k_3$.

IV. FUTURE TESTS

19. Future Tests

The ultimate objective of this investigation is an exhaustive study, both experimentally and analytically, of shear in prestressed concrete members. As the literature provided no significant information on this type of research, the tests of simply supported prestressed concrete beams with no web reinforcement constituted a logical exploratory series for the investigation. The I-Beam series was a foreseen sequel. After a satisfactory solution is provided to the question of the effect of cross-sectional geometry, tests shall presumably be extended to cover other variables such as web-reinforcement, sloped cables, continuity, etc.

The present status of the test data suggest the following to be carried out in the immediate future:

It is evident in Figs. 11a, 11b, and 11c that, especially for the 60,000 psi and 120,000 psi prestress series, the variation in Q is rather limited. Therefore, an effort to expand the range in both directions seems expedient. Previously lower values of Q had been purposely avoided to eliminate tension failures. However, it now appears that tests on a few low Q beams are desirable to establish a line of demarcation between shear and flexure failures. Casting beams with high values of Q will entail some manufacturing difficulties for the prestressed specimens as the excessive prestressing force is prone to detrimentally crack the beam before it is placed in the testing

machine. To alleviate such cracking the lengths of the specimens may have to be increased.

In Fig. 11c there are two low points pertaining to specimens with 1 3/4" webs. The relative weakness of these beams may be due to the thinness of their webs. More likely, however, is the possibility of these results being a caused by the cracks developed during fabrication. As it has already been alluded to, the large prestressing force causes the green specimen to crack if the concrete strength is low. Two types of cracks are developed. Tensile cracks, which traverse the top flange perpendicularly, close up on the application of load and do not affect the general behavior of the beam other than causing some erratic readings on the concrete strain gages. On the other hand, almost invisibly fine horizontal cracks which form at the junction of the web and the top flange, appear to be very detrimental to the load carrying capacity of the beam. If these hairline cracks exist in a beam, then, when a diagonal crack reaches their level, they suddenly open up and practically separate the top flange from the web. Presumably it is this action that causes distress in the web effecting a reduction in strength. The preceding is merely a surmise based on the observed state of two would be specimens which were scrapped when such cracks were detected on them. Specimen V6 which corresponds to the low point on Fig. 10b is one where these cracks were detected to a moderate degree prior to test. Its behavior under load was as explained above. Yet it cannot without reservation be upheld that the beams in the 120,000 psi series were so cracked

before test since no such observations were actually made. Consequently, at the present time it seems desirable to test a few beams simulating the properties of those which failed lower than what the general trend indicates.

In connection with the investigation of the effects of geometry of cross-section, it is also worthwhile to test a few I-Beams with depths greater than twelve inches.

After the satisfactory completion of the above tests and corresponding analyses, the next test series are intended to comprise specimens with varying amounts of web reinforcement.

V. SUMMARY

Tests have been completed on five rectangular beams and twenty-six I-Beams all simply supported over a span of 9'-0" and all reinforced in tension only.

The variables were the following: Concrete strength, percentage of longitudinal reinforcement, prestress, web thickness and shear span.

The measured ultimate capacities of the rectangular specimens were in striking agreement with the expressions derived by Billet and Zwoyer. Their behavior also implied that the mode of failure for a beam cannot be anticipated with absolute certainty if the computed value of Q' for that specimen is in the proximity of the critical value of Q' differentiating between shear and flexure failures. This value is that which predicts an ultimate steel stress higher than the yield stress in a simply supported prestressed concrete beam with no web reinforcement.

The general behavior of the test specimens during test to failure appears to validate the approach that the shear failures are compression failures occurring at a fraction of the ultimate flexural capacity of the beam, these immature compression failures being induced by diagonal cracks which restrict the flange area available to resist the compressive force.

The behavior under load of the I-Beams may be divided into three phases on basis of the crack pattern. The first phase corresponds to 'elastic' behavior that is observed when the beam is uncracked.

The appearance of the first tensile crack at mid-span marks the beginning of the 'inelastic' behavior and hence the second phase. The third phase is distinguished from the second by the existence of diagonal cracks which finally lead to failure.

There has been observed no significant disparity between the load-deflection relationships of I-Beams with 1 3/4" and 3" webs having all other variables practically identical. Both the ultimate load and deflection appeared to be direct functions of the prestress, the percentage of longitudinal reinforcement, and the concrete strength. Consistent with the concurrent explanation of shear failures, the ultimate load varied inversely with the shear span.

A favorable comparison has been obtained between observed and computed cracking loads. However, the practical value of this comparison is rather dubious as the computed cracking loads are based on actual moduli of rupture which, as seen in Fig. 5, exhibit a very erratic pattern.

Computed values of nominal shear and principal tension stresses for the test specimens once again emphasized the superfluousness of their calculation in connection with the shear strength of beams. As might be expected the former computed on basis of the ultimate shear varied in no consistent manner with any of the variables, and the latter, computed for an assumed working load, indicated unwarranted safety with respect to shear.

Plots of $\frac{M_{ult}}{f'_c\, bd^2}$ versus Q showed trends favorable to the derivation of an analytical expression modified by an empirical factor $K\epsilon_u$ as had been done for the rectangular beams. However, attempts to evaluate this factor, which in effect relates the steel strain to the concrete strain at the instant of failure, directly from test data proved unsuccessful (Fig. 10). Therefore an analytical study of the variation of the ultimate shear capacity with $K\epsilon_u$ was made. This disclosed that the sensitivity of the ultimate moment to changes in $K\epsilon_u$ was quite low, which meant that a slight error in the measuring of the quantity $\frac{M}{k_1 k_3 f'_c\, bd^2}$ would be magnified in $K\epsilon_u$ if the latter is evaluated from the test moment. At present it appears feasible to establish $K\epsilon_u$ by a trial and error procedure disregarding the unreliably measured values of the depth of the neutral axis at failure.

The next step in the experimental program is the testing of several I-Beams suggested by the present status of the test data. Most of these beams shall have extreme values of the ratio of amount of steel to concrete strength, and some shall probably have different spans and overall depths. Testing of beams with varying amounts of web reinforcement are also contemplated subsequent to a satisfactory completion of the present series.

TABLE 1

PROPERTIES OF SPECIMENS

Mark	Concrete Strength f'_c psi	*Type of Reinforcement	Flange Width b ins.	Web Thickness b' ins.	Effective Depth d ins.	Area of steel A_s sq. ins.	Longit. Reinf. p %	Effective Prestress f_{se} psi	Shear Span a ins.
				Rectangular Beams					
S-2a	3480	I	6.10	----	8.75	0.1746	0.327	49,300	36
V1	4410	I	6.08	----	8.94	0.0873	0.161	0	36
V2	4180	I	6.05	----	9.24	0.0582	0.104	0	36
V3	3810	I	6.04	----	8.85	0.1164	0.218	0	36
V4	4990	I	6.15	----	9.03	0.1746	0.314	0	36
				I-Beams					
V5	2060	I	6.10	1.89	9.00	0.2328	0.424	54,300	36
V6	2150	I	6.15	1.85	10.18	0.1746	0.279	54,500	36
V7	2490	I	6.19	1.84	10.40	0.1164	0.181	60,000	36
V8	3060	I	6.14	1.83	10.01	0.2328	0.377	0	36
V9	3870	I	6.08	1.82	10.00	0.1746	0.287	0	36
V10	2690	I	6.14	1.88	10.10	0.2328	0.375	0	36
V11	3620	I	6.17	1.86	9.86	0.2328	0.383	103,000	36
V12	5470	II	6.11	1.88	10.08	0.3732	0.606	115,400	36
V13	6040	VI	6.00	1.79	10.11	0.2328	0.384	111,100	36
V14	2770	VI	6.15	3.11	10.15	0.1746	0.280	56,700	36
V15	2710	VI	6.25	3.16	10.02	0.2328	0.372	51,200	36
V16	1750	VI	6.20	3.12	9.95	0.2328	0.377	59,900	36
V17	4330	VI	6.16	3.12	10.21	0.1746	0.278	0	36
V18	2720	VI	6.19	3.10	10.20	0.1746	0.277	0	36
V19	2270	I	6.19	3.11	10.08	0.2328	0.373	0	36
V20	4420	VI	6.14	3.03	10.06	0.2328	0.377	110,000	36
V21	4825	VI	6.19	3.08	10.18	0.3492	0.554	107,400	36
V22	4180	VII	6.00	3.00	9.76	0.2376	0.406	121,700	36
V23	2030	VI	6.16	3.08	10.05	0.2328	0.376	60,200	36
V24	2510	VII	6.26	3.20	10.11	0.1782	0.382	0	36
V25	3210	VII	6.30	3.08	9.99	0.2376	0.378	121,000	36
V26	5120	VII	6.05	3.00	10.03	0.2376	0.391	55,300	36

TABLE 1 (cont.)

PROPERTIES OF SPECIMENS

Mark	Concrete Strength f'_c psi	*Type of Reinforcement	Flange Width b ins.	Web Thickness b' ins.	Effective Depth d ins.	Area of steel A_s sq. ins.	Longit. Reinf. p %	Effective Prestress f_{se} psi	Shear Span a ins.
V27	2060	VII	6.04	3.00	10.45	0.1188	0.188	122,000	36
V28	5820	VII	5.98	2.95	10.21	0.1782	0.292	0	54
V29	4470	VII	6.00	2.96	10.06	0.2376	0.393	62,300	54
V30	4525	VII	5.92	2.95	10.21	0.1782	0.295	123,500	54

*Refer to Section 4d for description of reinforcement.

TABLE 2

SIEVE ANALYSIS OF AGGREGATES

Sieve No.	Percentage Retained	
	Sand	Gravel
3/8	----	0
4	0.3	96.0
8	4.4	99.2
16	26.8	99.3
30	62.1	99.3
50	91.3	99.4
100	98.3	----
Fineness Modulus	2.83	

TABLE 3

PROPERTIES OF CONCRETE MIXES

Beam	Cem:Sand:Gravel by weight	Water/Cement		Slump in.		Concrete Strength f'_c psi		Age
		Batch		Batch		Batch		
		I	II	I	II	I	II	
S-2a	1:3.9:5.5	1.19	1.19	3	6	3770	3480	36
V1	1:3.4:5.2	1.30	1.35	2	2	4430	4410	24
V2	1:3.4:5.2	1.35	1.43	2	1 1/2	4000	4180	22
V3	1:3.5:5.1	1.43	1.41	1	2	4120	3810	21
V4	1:2.8:4.3	1.52	1.59	2 1/2	2	4810	4990	19
V5	1:4.0:4.0	0.91	0.94	7	8	2510	2060	6
V6	1:5.0:5.0	1.07	0.97	1	1/2	1030	2310	8
V7	1:4.4:4.5	1.05	1.04	1 1/2	3 1/2	2270	2490	7
V8	1:4.5:4.6	0.92	0.92	1	1 1/2	3630	3060	6
V9	1:4.0:4.0	0.85	0.82	1 1/2	1 1/2	3920	3870	8
V10	1:4.5:4.5	1.07	1.02	5 1/2	7 1/2	2610	2690	13
V11	1:4.0:4.0	0.84	0.83	2	1 1/2	1880	3620	7
V12	1:2.7:3.0	0.64	0.61	2 1/2	2 1/2	5390	5470	7
V13	1:2.7:3.0	0.67	0.65	5	3 1/2	5890	6040	12
V14	1:4.4:4.4	0.95	0.95	3	4	3290	2770	7
V15	1:4.6:4.6	0.93	0.95	1	6	2750	2710	10
V16	1:4.7:4.7	1.10	1.10	6	6 1/2	1770	1750	10
V17	1:3.3:3.5	0.74	0.74	4	5	4580	4330	12
V18	1:4.7:4.7	0.94	0.94	3	6	2620	2720	8
V19	1:4.7:4.7	1.00	0.98	2 3/4	2 1/2	2450	2270	8
V20	1:3.6:3.7	0.71	0.69	2	2	4460	4420	9
V21	1:2.6:2.9	0.72	0.69	8	8	5140	4825	8
V22	1:3.7:3.6	0.72	0.74	1	2	4400	4180	8
V23	1:4.6:4.7	1.05	1.05	1 1/2	1 1/2	2230	2030	8
V24	1:4.7:4.7	1.08	1.08	6 1/2	6 1/2	2360	2510	13
V25	1:4.6:4.5	0.91	0.91	3	3	3240	3210	13
V26	1:3.3:3.5	0.70	0.70	2	2 1/2	5160	5120	14
V27	1:4.2:4.2	1.00	1.00	7	6 1/2	3000	2060	8
V28	1:3.0:3.3	0.61	0.61	2	2	5650	5820	9
V29	1:3.3:3.4	0.75	0.75	4	4	4320	4470	7
V30	1:3.3:3.4	0.70	0.70	3 1/2	3 1/2	4650	4525	6

TABLE 4

COMPUTED AND OBSERVED CRACKING LOADS

Mark	Area of Steel A_s	Effective Prestress f_{se}	Effective Prestress Force F	Modulus of Rupture f_r	Cracking Load P_{cr} Comp.	Cracking Load P_{cr} Obs.	Obs. P_{cr} / Comp. P_{cr}
	sq. in.	psi	lb.	psi	lb.	lb.	
S-2a	0.1746	49,300	8,610	460	5,460	5,570	1.02
V1	0.0873	0	0	600	4,300	-----	----
V2	0.0582	0	0	480	3,340	2,850	0.85
V3	0.1164	0	0	490	3,420	2,750	0.80
V4	0.1746	0	0	550	3,900	4,250	1.09
V5	0.2328	54,300	12,640	240	5,420	4,500	0.83
V6	0.1746	54,500	9,520	240*	4,970	4,300	0.87
V7	0.1146	60,000	6,900	330	4,780	4,200	0.88
V8	0.2328	0	0	400*	2,500	2,600	1.04
V9	0.1746	0	0	470	3,020	2,850	0.94
V10	0.2328	0	0	350	2,130	1,500	0.70
V11	0.2328	103,000	24,000	250*	10,280	9,500	0.92
V12	0.3732	115,400	43,060	480	16,840	16,000	0.95
V13	0.2328	111,100	25,860	400	12,480	12,000	0.96
V14	0.1746	56,700	9,900	500	6,910	6,000	0.87
V15	0.2328	51,200	11,920	300	6,060	6,000	0.99
V16	0.2328	59,900	13,940	220	6,850	6,000	0.88
V17	0.1746	0	0	320	1,900	2,300	1.21
V18	0.1746	0	0	250	1,390	1,500	1.08
V19	0.2328	0	0	290	1,680	1,500	0.89
V20	0.2328	110,000	25,600	300	11,030	9,800	0.89
V21	0.3492	107,400	37,500	380	16,200	15,800	0.98
V22	0.2376	121,700	28,920	430	12,700	12,500	0.98
V23	0.2328	60,200	14,000	340	7,110	4,500	0.63
V24	0.1782	0	0	300	1,760	1,500	0.85
V25	0.2376	121,000	28,750	400	12,760	12,900	1.01
V26	0.2376	55,300	13,140	390	7,140	6,000	0.84
V27	0.1188	122,000	14,490	420	8,180	6,500	0.80
V28	0.1782	0	0	450	1,910	1,900	0.99
V29	0.2376	62,300	14,800	510	5,780	5,900	1.02
V30	0.1782	123,500	22,000	510	7,640	8,250	1.08
						Average =	0.93

*Values assumed according to Fig. 5.

TABLE 5

NOMINAL SHEAR STRESSES AND PRINCIPAL TENSION STRESSES

Mark	Concrete Strength f'_c psi	Longitudinal Steel p	a/d	Ultimate Shear V_{ult} lb.	Nominal Ultimate Shear $v = \frac{V_{ult}}{bjd}$ psi	$\frac{v}{f'_c}$	Max. Principal Tensile Stress at Design Load S_t psi	$\frac{S_t}{f'_c}$
S-2a	3480	0.327	4.1	6,440	123.1	.035		
V1	4410	0.161	4.0	4,930	92.6	.021		
V2	4180	0.104	3.9	3,540	64.6	.016		
V3	3810	0.218	4.1	6,075	116.0	.030		
V4	4990	0.314	4.0	6,010	110.8	.022		
V5	2060	0.424	4.0	5,340	103.7	.050		
V6	2150	0.279	3.5	3,830	62.8	.029		
V7	2490	0.181	3.5	4,840	77.8	.031		
V8	3060	0.377	3.6	4,860	81.2	.027		
V9	3870	0.287	3.6	5,390	90.6	.023		
V10	2690	0.375	3.6	4,290	71.7	.027		
V11	3620	0.383	3.7	8,370	148.3	.041	173	.048
V12	5470	0.606	3.6	12,970	221.2	.040	337	.062
V13	6040	0.384	3.6	11,390	196.2	.033	198	.033
V14	2770	0.280	3.6	7,720	129.4	.047		
V15	2710	0.372	3.6	8,920	151.2	.056		
V16	1750	0.377	3.6	5,670	96.7	.055		
V17	4330	0.278	3.5	5,470	89.2	.021		
V18	2720	0.277	3.5	4,020	64.8	.024		
V19	2270	0.373	3.6	3,840	62.9	.028		
V20	4420	0.377	3.6	11,870	211.6	.048	103	.023
V21	4825	0.554	3.5	14,590	250.3	.052	98	.020
V22	4180	0.406	3.7	12,720	235.6	.056	110	.026
V23	2030	0.376	3.6	5,780	96.5	.048		
V24	2510	0.282	3.6	5,050	81.8	.033		
V25	3210	0.378	3.6	11,600	200.3	.062	116	.036
V26	5120	0.391	3.6	9,520	160.2	.031		
V27	2060	0.188	3.4	6,390	105.5	.051	0	--
V28	5820	0.292	3.5	4,620	75.7	.013		
V29	4470	0.393	3.6	6,370	109.8	.025		
V30	4525	0.295	3.5	7,070	120.9	.027	0	--

TABLE 6

MEASURED AND COMPUTED ULTIMATE CAPACITY FOR THE RECTANGULAR SPECIMENS

Mark	$\frac{E_s p^*}{k_1 k_3 f'_c}$		Effective Prestress	$\frac{\text{Mult}}{k_1 k_3 f'_c b d^2}$			$\frac{M_{meas.}}{M_{flex.}}$	$\frac{M_{meas.}}{M_{shear}}$	Mode** of Fail.	Type Reinf.***
	Billet	Zwoyer	psi	Meas.	Comp. for Flex.	Comp. for Shear				
S-2a		22.6	49,300	.118		.122	--	0.97	S	I
V1	12.4	9.2	0	.092	.094		0.98	--	T	I
V2	8.3	6.2	0	.064	.064		1.00	--	T	I
V3	18.7	14.1	0	.130	.129		1.01	--	T	I
V4		16.5	0	.074		.075	--	0.99	S	I

* Billet - $k_1 k_3 = \frac{3000 + 1/2\ f'_c}{1500 + f'_c}$

Zwoyer - $k_1 k_3 = 1.50 - 0.715 \times 10^{-4} f'_c$

** T-Tension
S-Shear

*** Refer to Section 4d for description of reinforcement.

TABLE 7

MEASURED ULTIMATE CAPACITY, k_1k_3 and k_u

Mark	Concrete Strength f'_c psi	Longitudinal Steel p %	$\frac{E_s p}{f'_c}$ Q	Effective Prestress f_{se} psi	Measured Depth to Neutral Axis k_u	Ultimate Load P_{ult} lb.	a/d	$\frac{M_{ult}}{f'_c bd^2}$	$K\epsilon_u$ 10^5	k_1k_3
S-2a	3480	.327	28.2	49,300	0.114	12,880	4.0	0.148	46.6	1.31
V1	4410	.161	11.0	0	0.084	9,180	4.0	0.081	--	0.98
V2	4180	.104	7.5	0	0.103	6,400	3.9	0.058	--	0.54
V3	3810	.218	17.2	0	0.085	11,470	4.1	0.120	--	0.83
V4	4990	.314	18.9	0	0.069	11,340	4.0	0.085	33.0	1.21
V5	2060	.424	61.7	54,300	0.146	10,250	4.0	0.187	20.5	1.32
V6	2150	.279	38.9	54,500	0.061	7,220	3.5	0.099	3.59	1.58
V7	2490	.181	21.8	60,000	0.078	9,250	3.5	0.104	22.6	1.33
V8	3060	.377	37.0	0	0.062	9,280	3.6	0.092	16.2	1.46
V9	3870	.287	22.2	0	0.050	10,350	3.6	0.082	20.0	1.62
V10	2690	.375	41.8	0	0.084	8,150	3.6	0.091	19.9	1.08
V11	3620	.383	31.7	103,000	0.171	16,300	3.7	0.138	5.2	0.85
V12	5470	.606	33.2	115,400	0.115	25,500	3.6	0.138	0	1.36
V13	6040	.384	19.1	111,100	0.103	22,340	3.6	0.110	25.4	1.10
V14	2770	.280	30.3	56,700	0.104	15,000	3.6	0.157	38.4	1.54
V15	2710	.372	41.2	51,200	0.137	17,500	3.6	0.187	46.2	1.42
V16	1750	.377	64.6	59,900	---	10,900	3.6	0.188	--	--
V17	4330	.278	19.3	0	0.061	10,500	3.5	0.069	23.5	1.15
V18	2720	.277	30.6	0	0.043	7,600	3.5	0.080	11.7	1.85
V19	2270	.373	49.3	0	0.050	7,250	3.6	0.095	9.9	1.88
V20	4420	.377	25.6	110,000	0.218	23,300	3.6	0.155	73.9	0.77
V21	4825	.554	34.5	107,400	0.178	28,750	3.5	0.169	28.4	1.02
V22	4180	.406	29.1	121,700	0.185	25,100	3.7	0.191	13.1	1.11
V23	2030	.376	55.6	60,200	0.075	11,120	3.6	0.163	2.1	2.19
V24	2510	.282	33.7	0	0.059	9,670	3.6	0.112	21.5	1.88
V25	3210	.378	35.3	121,000	0.190	22,780	3.6	0.206	45.3	1.16
V26	5120	.391	22.9	55,300	0.058	18,600	3.6	0.109	18.3	1.90
V27	2060	.188	27.4	122,000	0.096	12,350	3.4	0.167	20.3	1.78
V28	5820	.292	15.1	0	---	8,800	3.5	0.067	--	--
V29	4470	.393	26.4	62,300	0.090	12,300	3.6	0.124	21.5	1.42
V30	4525	.295	19.6	123,500	0.076	13,700	3.5	0.134	23.5	1.79

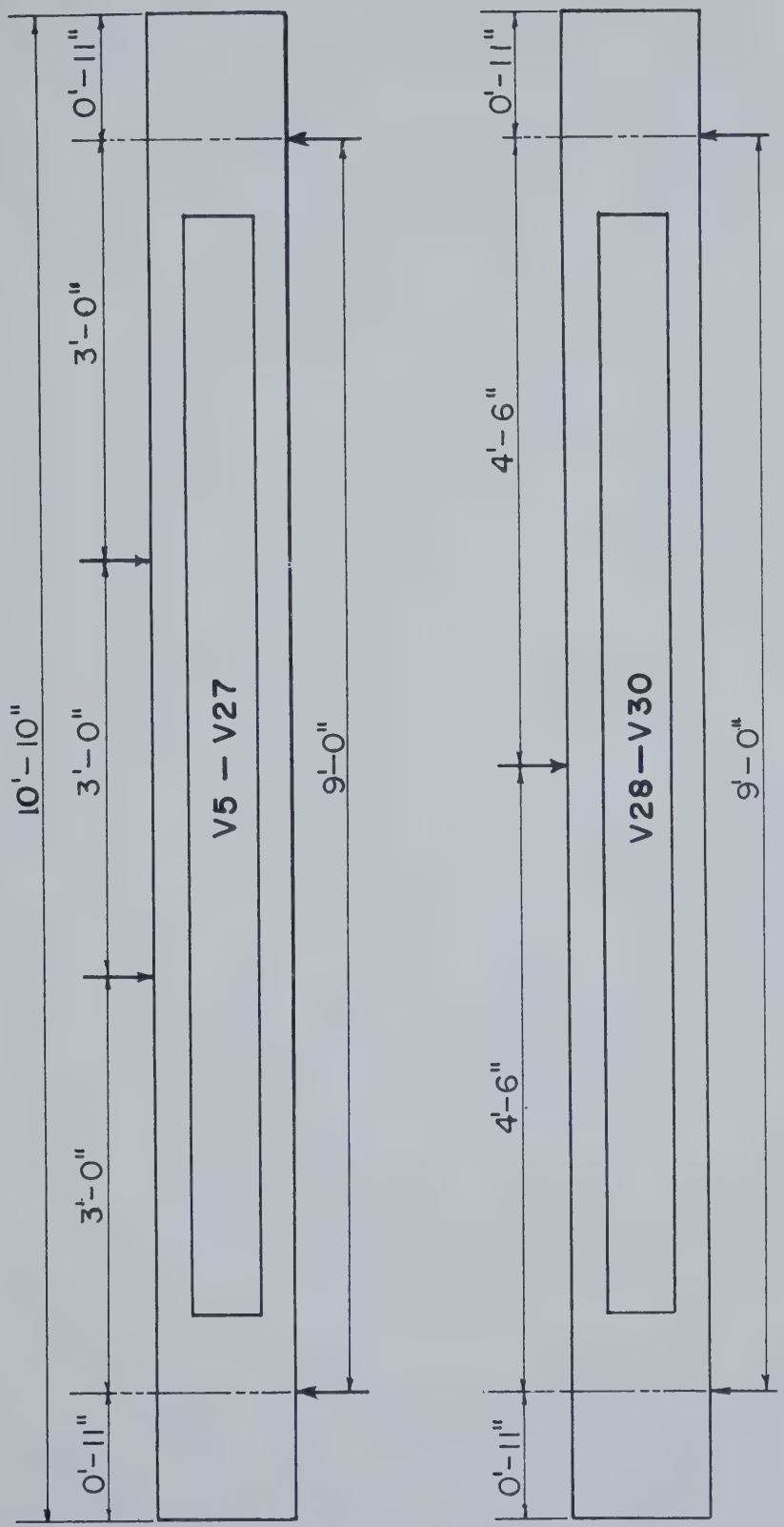
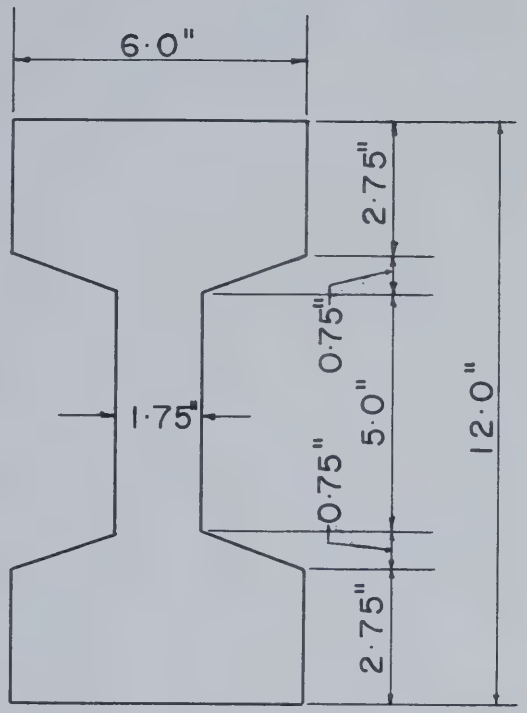
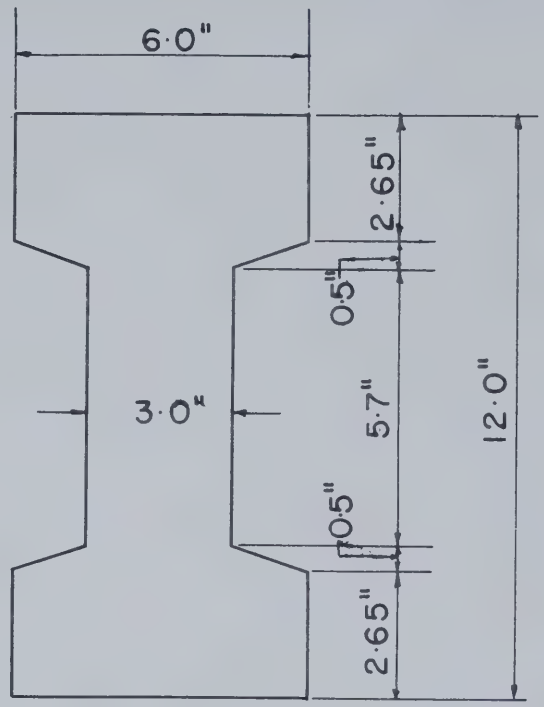


FIG. 1a DIMENSIONS AND LOADING ARRANGEMENT FOR I-BEAMS



V5 — V13



V14 — V30

FIG.1b NOMINAL DIMENSIONS FOR I-BEAMS

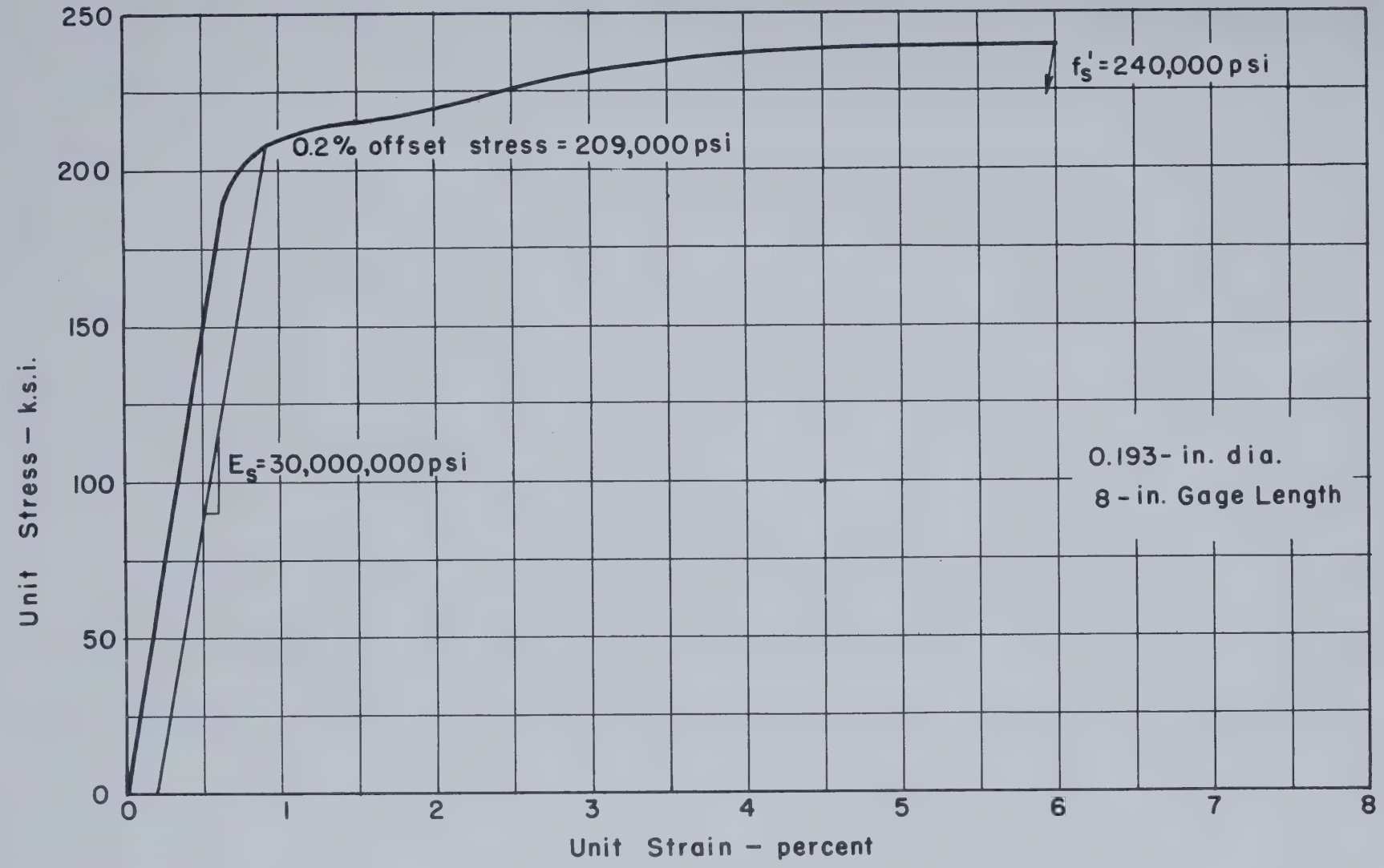


FIG. 2a STRESS-STRAIN RELATIONSHIP FOR TYPE I STEEL

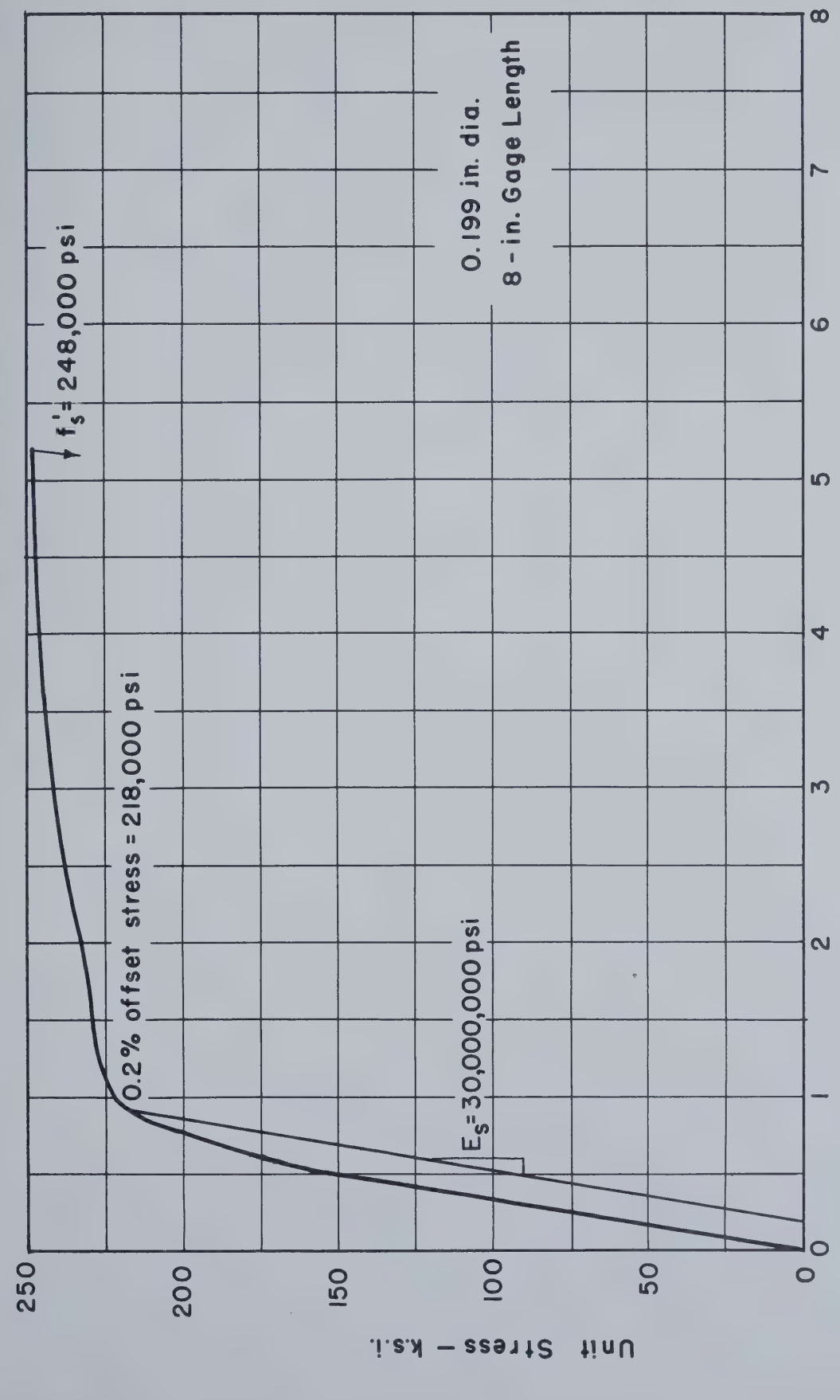


FIG. 2b STRESS-STRAIN RELATIONSHIP FOR TYPE II STEEL

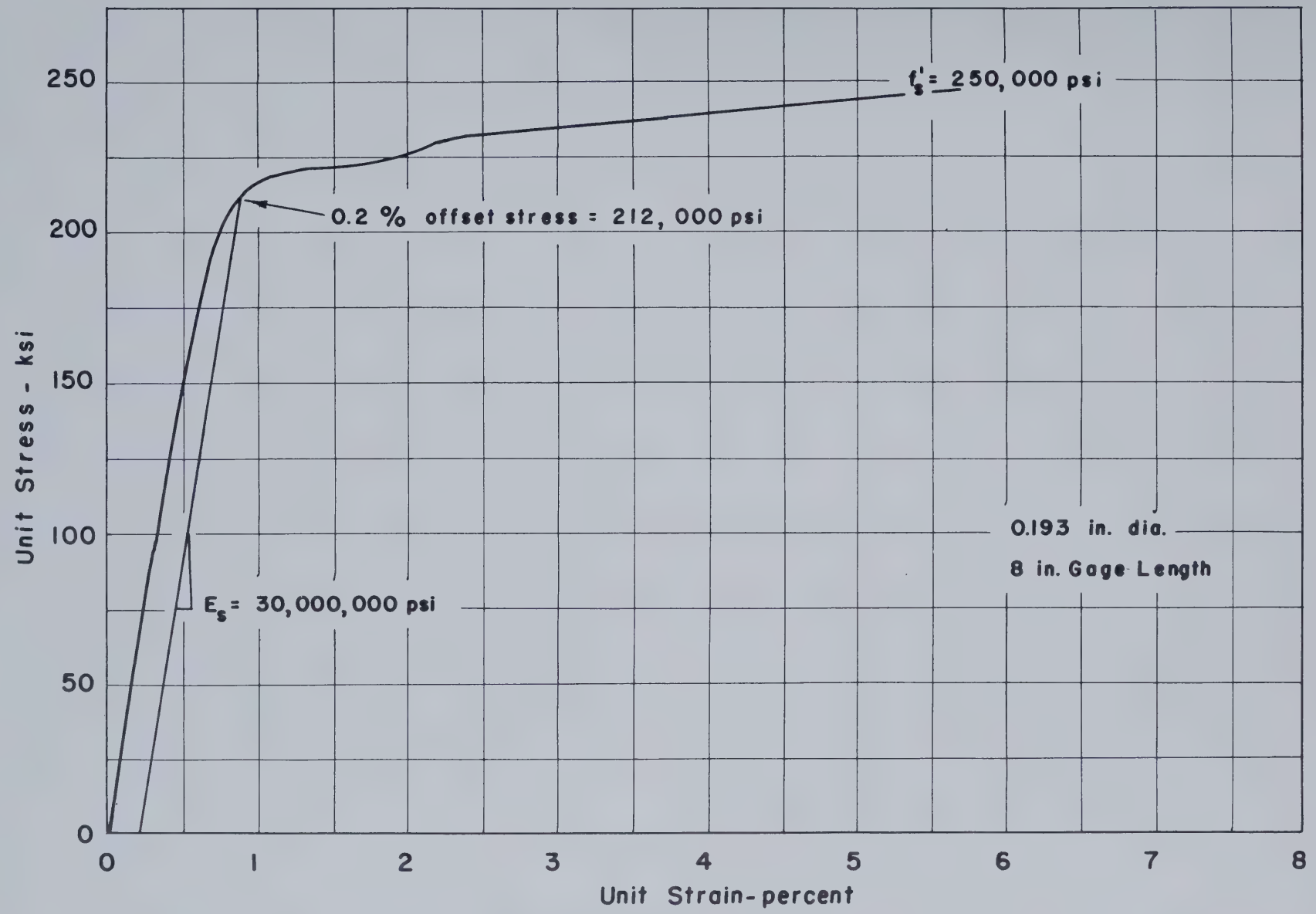


FIG. 2c STRESS-STRAIN RELATIONSHIP FOR TYPE VI STEEL

CH8.

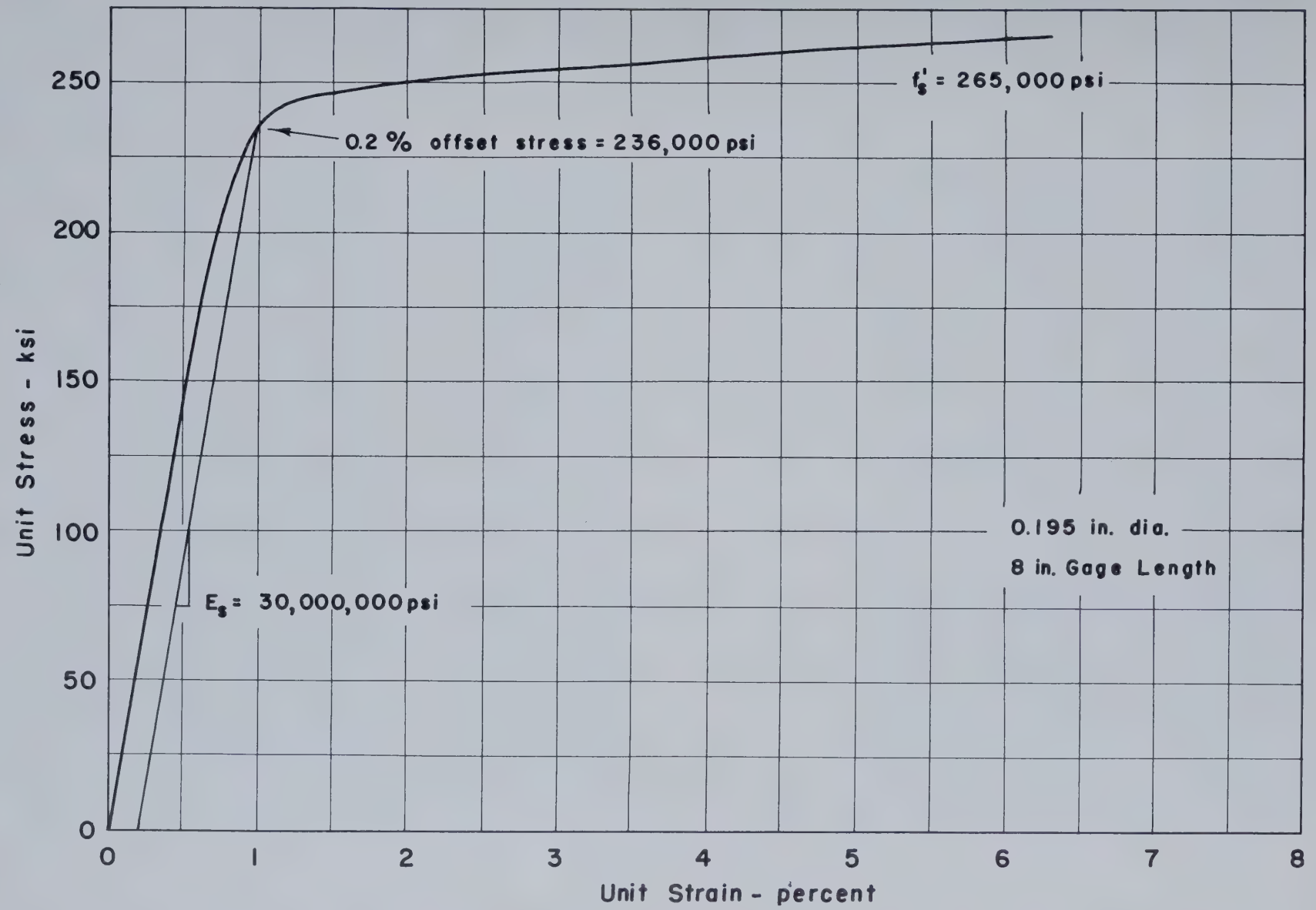


FIG. 2d STRESS-STRAIN RELATIONSHIP FOR TYPE VII STEEL

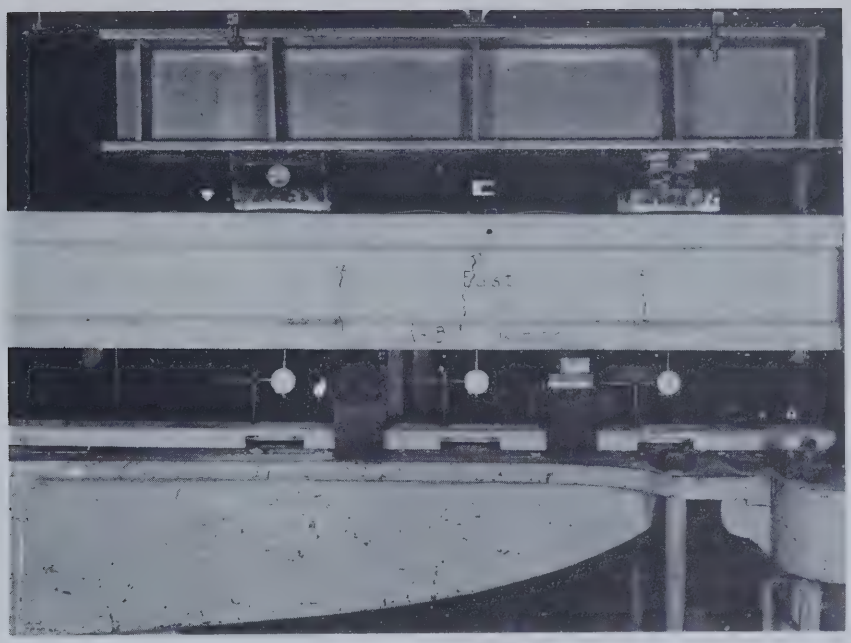


Fig. 3a Test Specimen Before Appearance of Diagonal Cracks

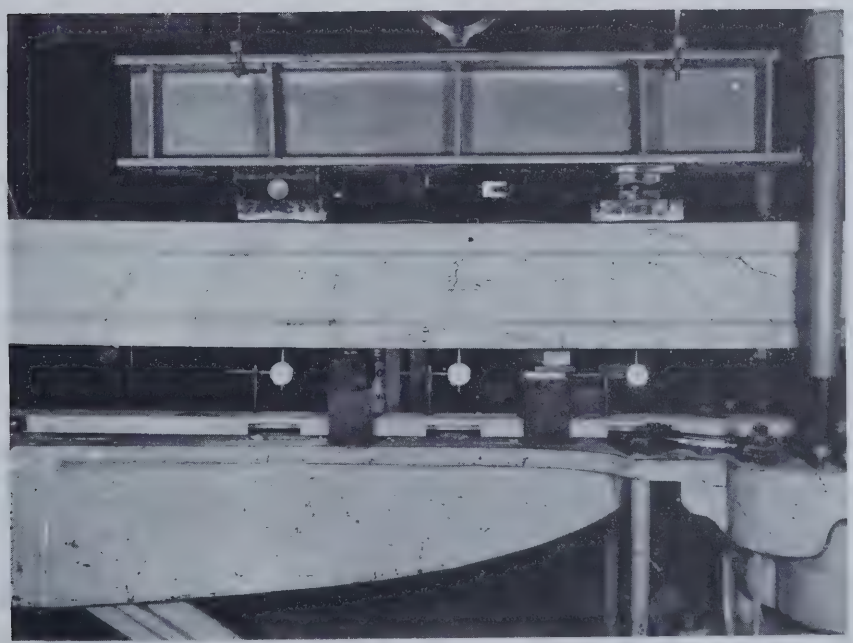
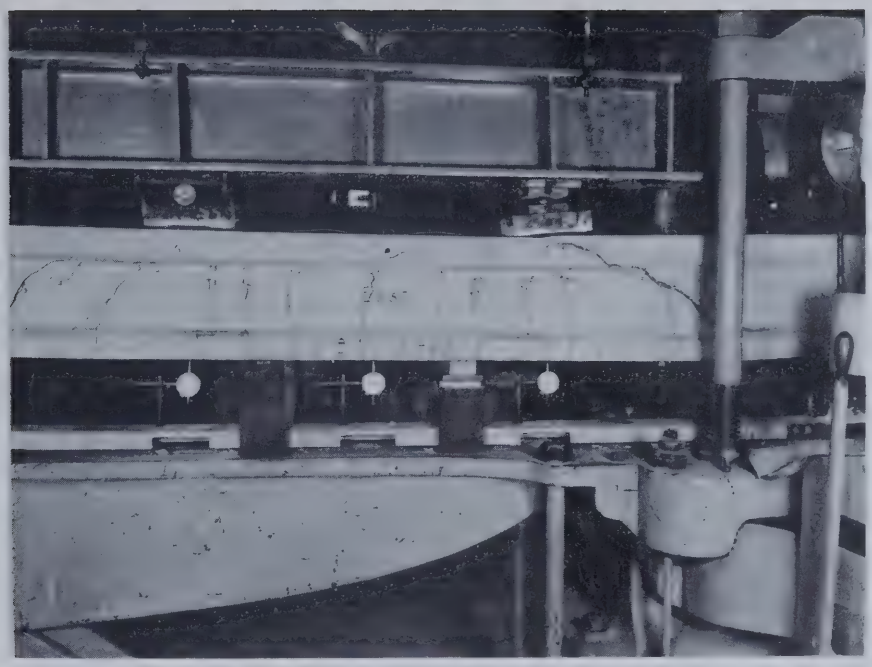


Fig. 3b Test Specimen After Appearance of Diagonal Cracks



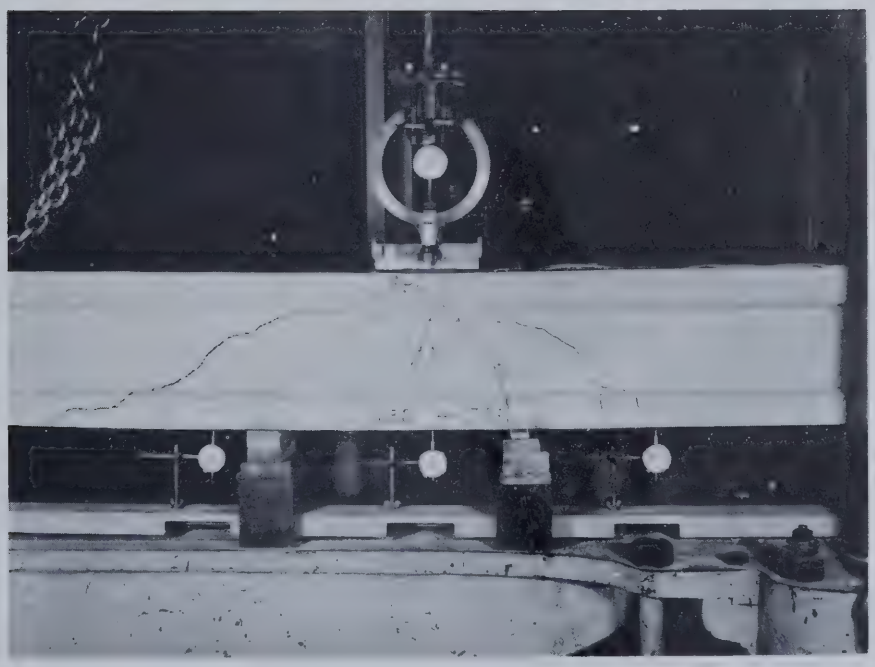


Fig. 3c Typical Shear Failures

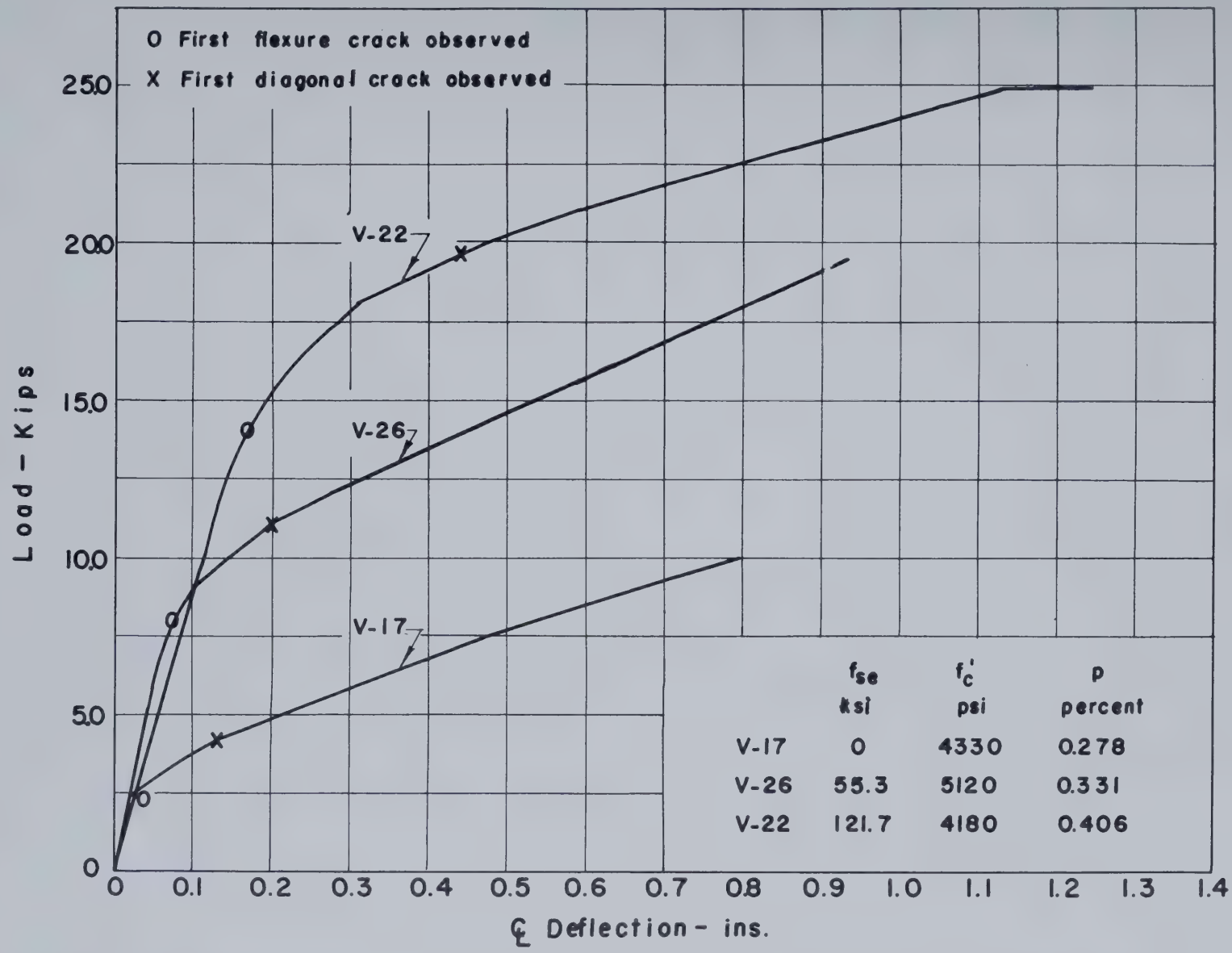


FIG. 4a COMPARISON OF LOAD-DEFLECTION RELATIONSHIPS AT DIFFERENT PRESTRESSES

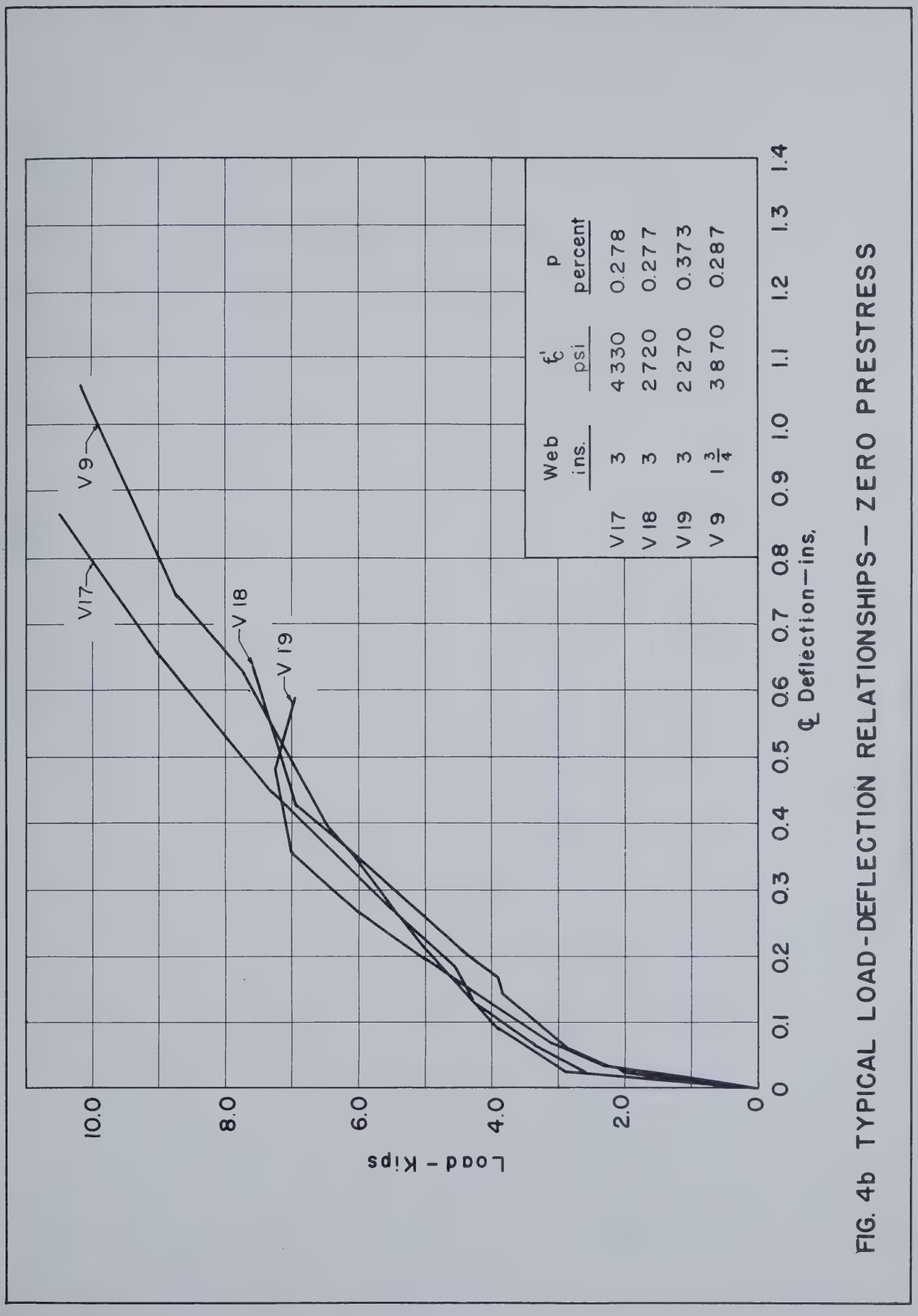
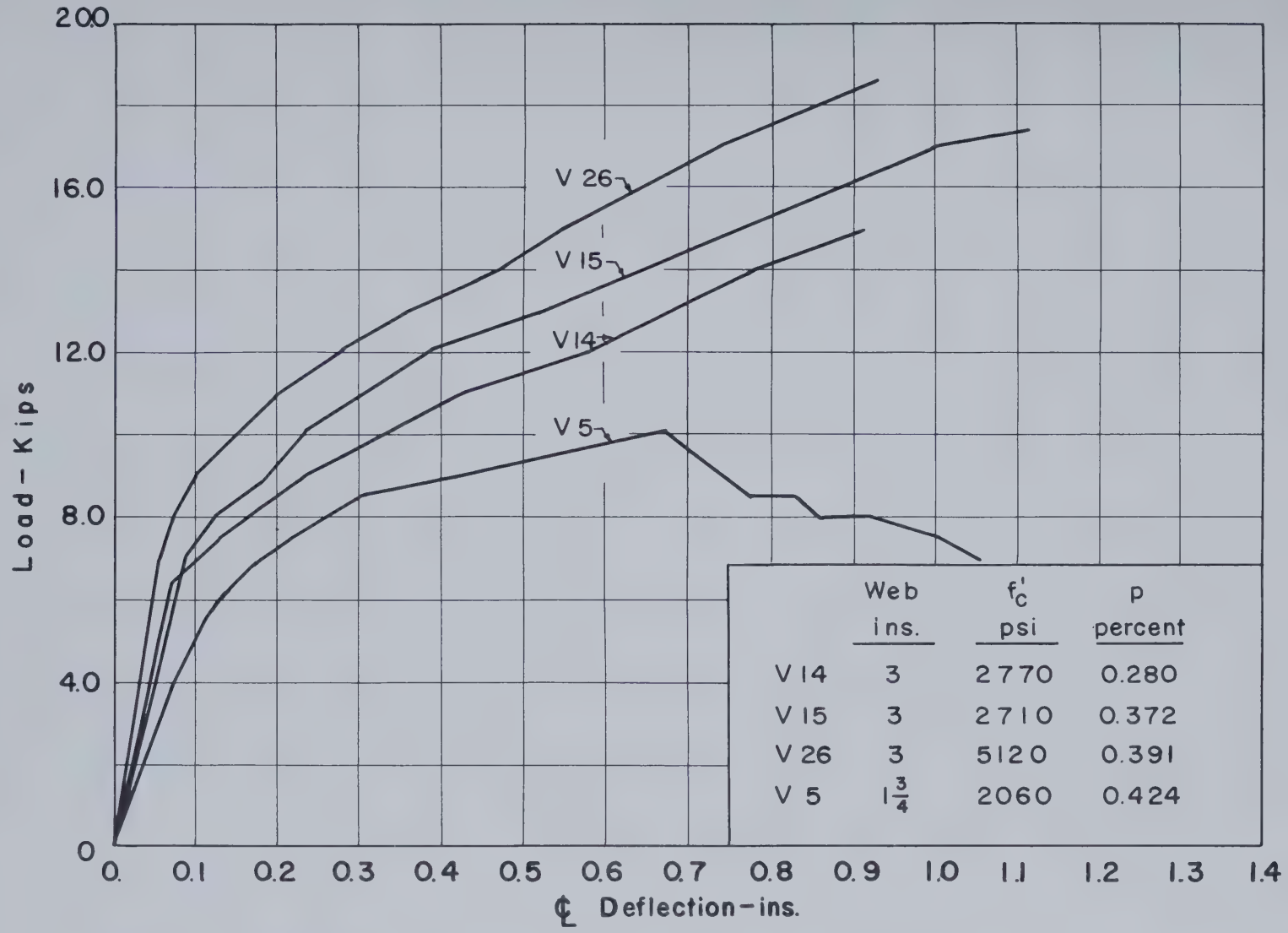


FIG. 4b TYPICAL LOAD-DEFLECTION RELATIONSHIPS— ZERO PRESTRESS



	Web ins.	f'_c psi	p percent
V 14	3	2770	0.280
V 15	3	2710	0.372
V 26	3	5120	0.391
V 5	$1\frac{3}{4}$	2060	0.424

FIG. 4c TYPICAL LOAD-DEFLECTION RELATIONSHIPS

NOMINAL PRESTRESS 60, 000 psi

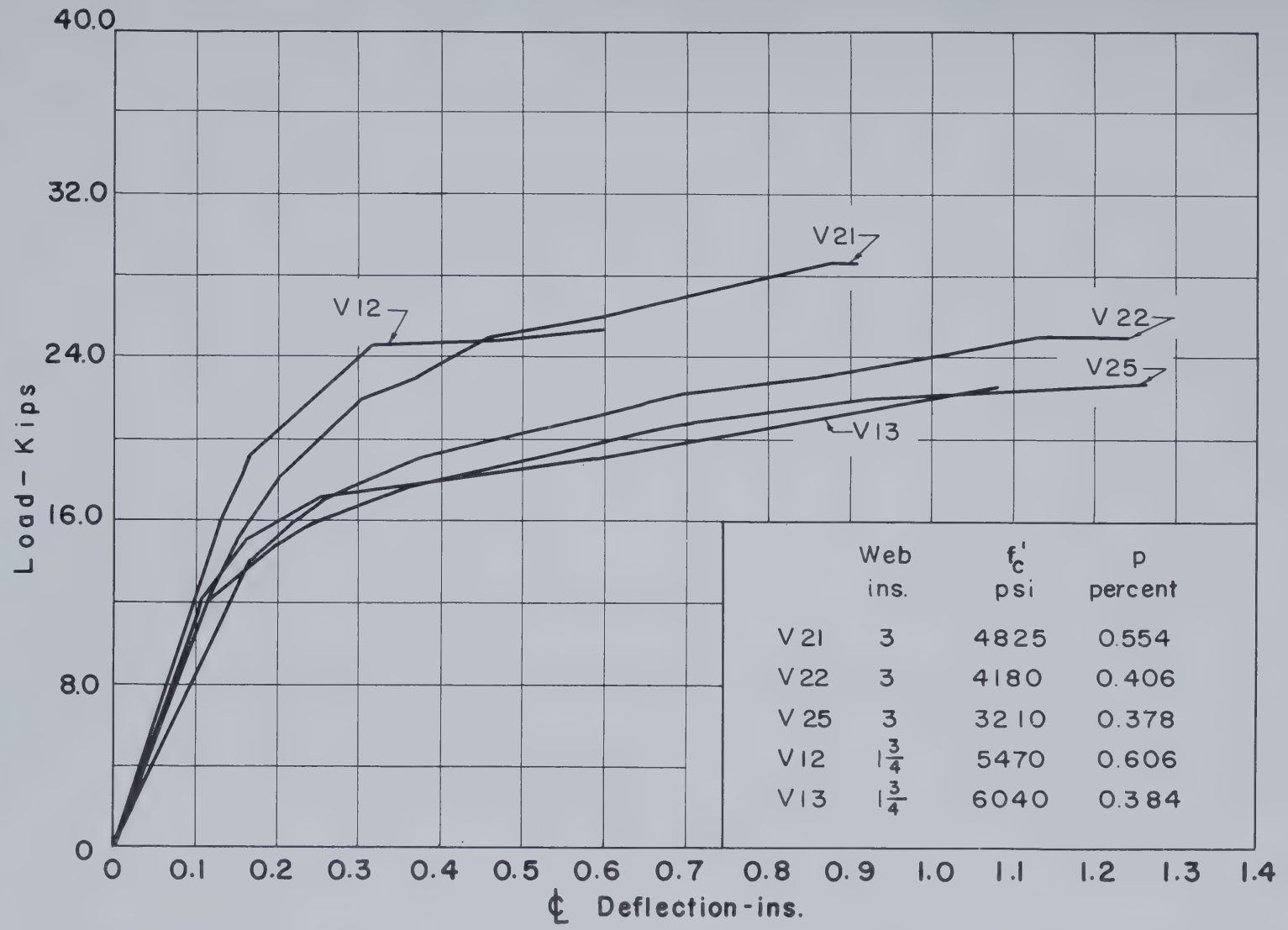


FIG. 4d TYPICAL LOAD-DEFLECTION RELATIONSHIPS

NOMINAL PRESTRESS 120,000 psi

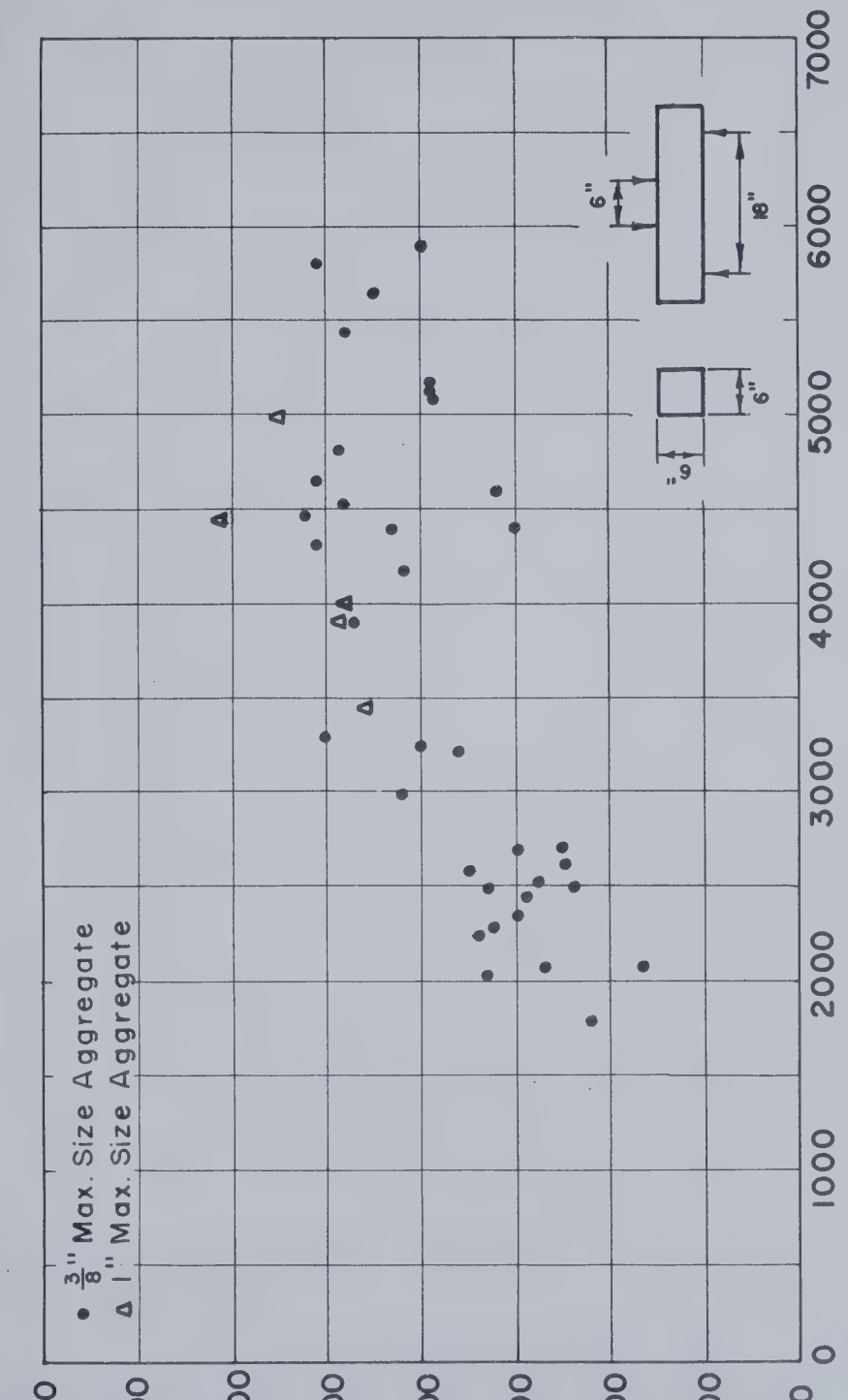


FIG. 5 COMPARISON OF MODULUS OF RUPTURE TO CONCRETE STRENGTH

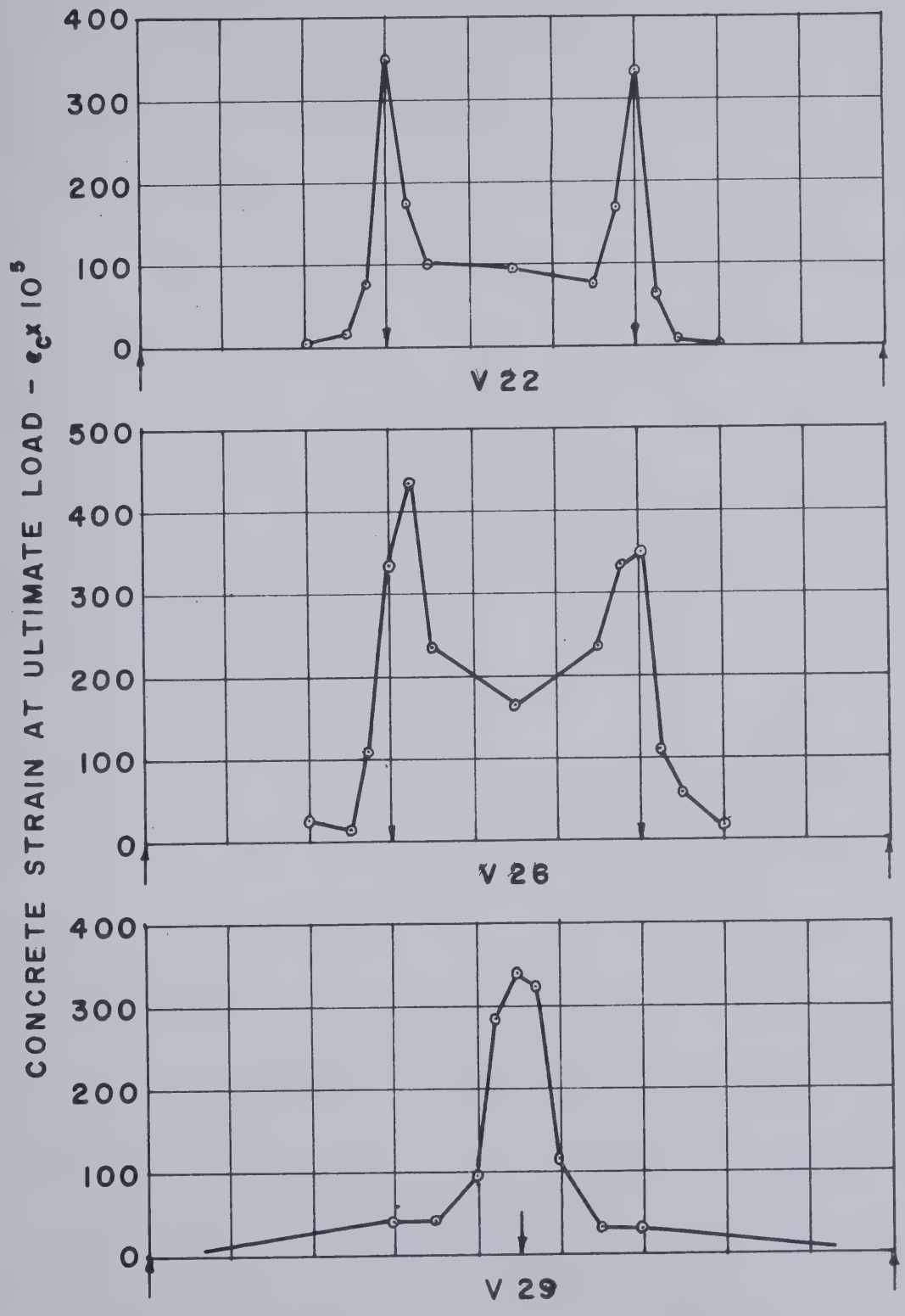


FIG. 6 DISTRIBUTION OF STRAIN AT ULTIMATE LOAD ALONG TOP SURFACE OF BEAMS FAILING IN SHEAR

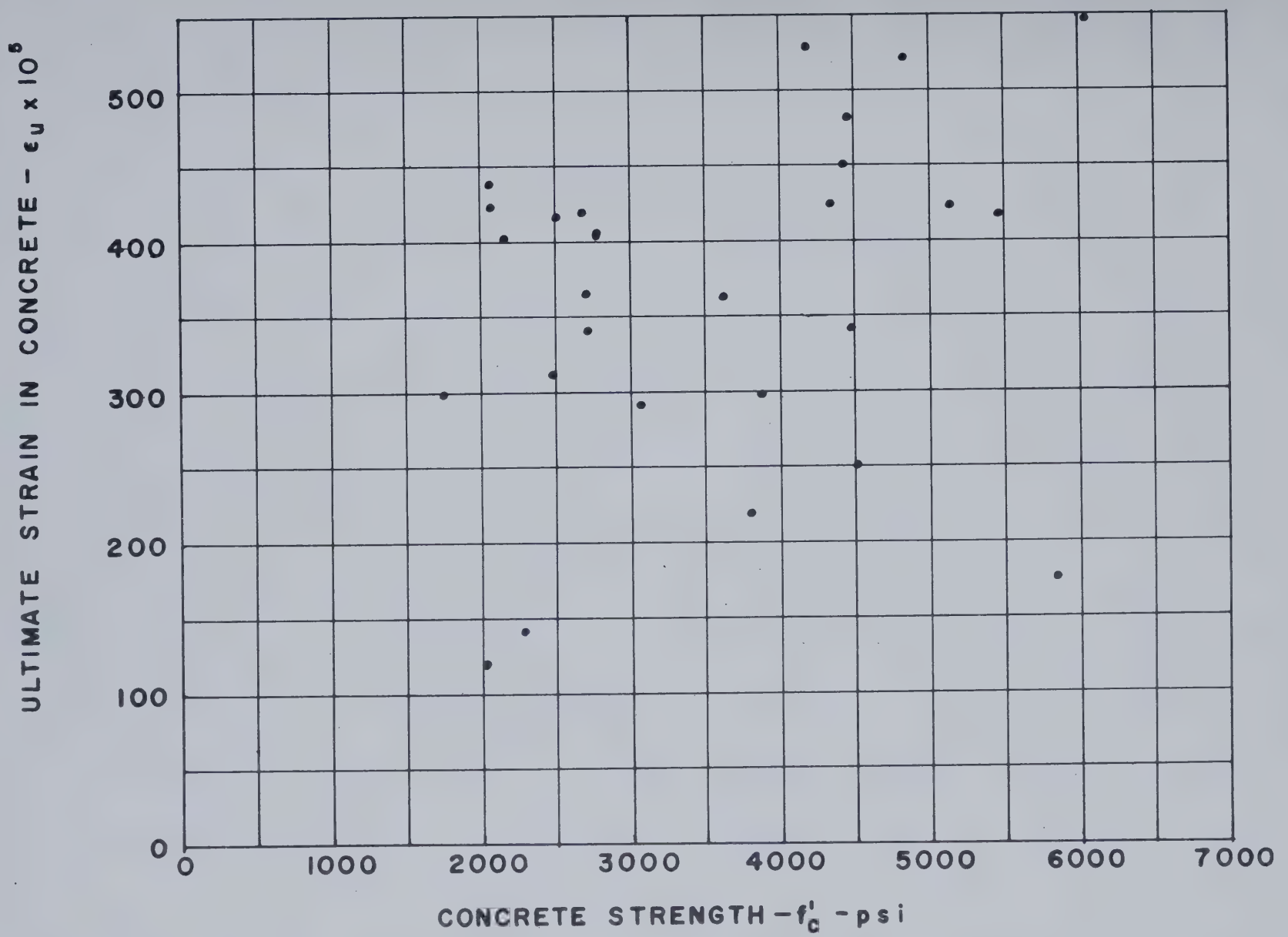


FIG. 7 COMPARISON OF ULTIMATE CONCRETE STRAIN WITH CONCRETE STRENGTH

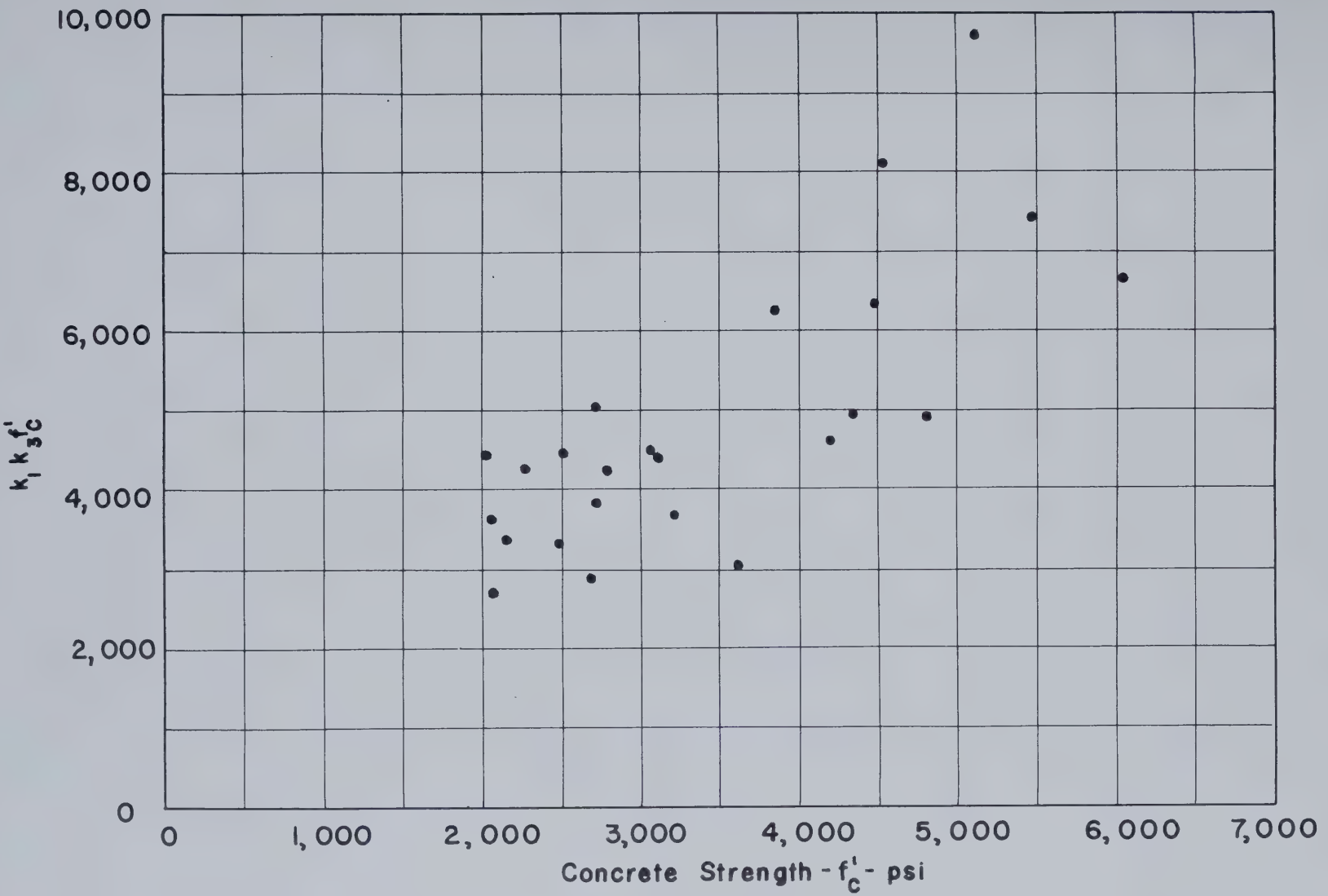


FIG. 8 COMPARISON OF $k_1 k_3 f'_c$ WITH f'_c

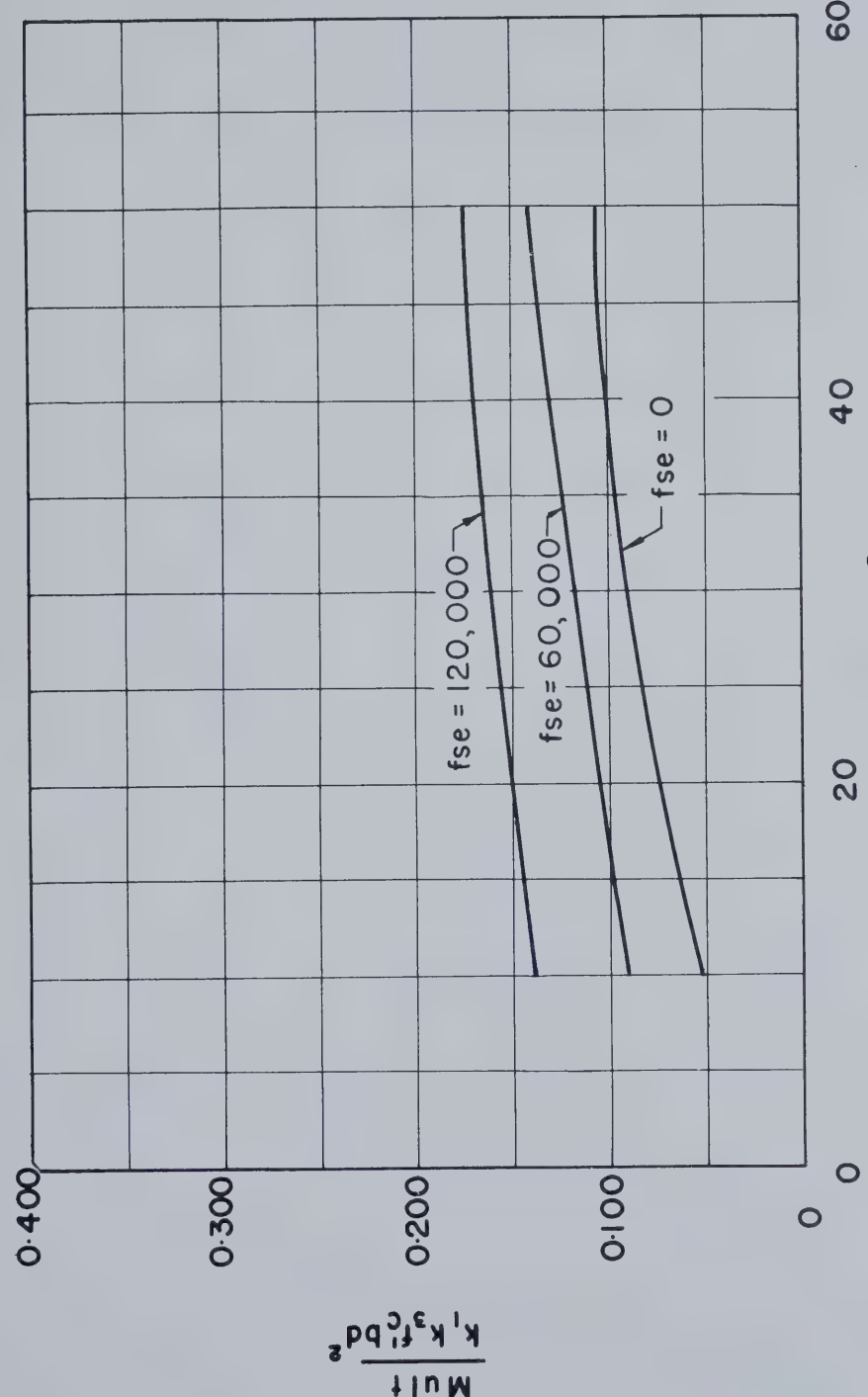


FIG. 9 VARIATION OF $\frac{Mult}{k_1 k_3 f'_c b d^2}$ WITH $K\epsilon_u$ FOR Q = 30

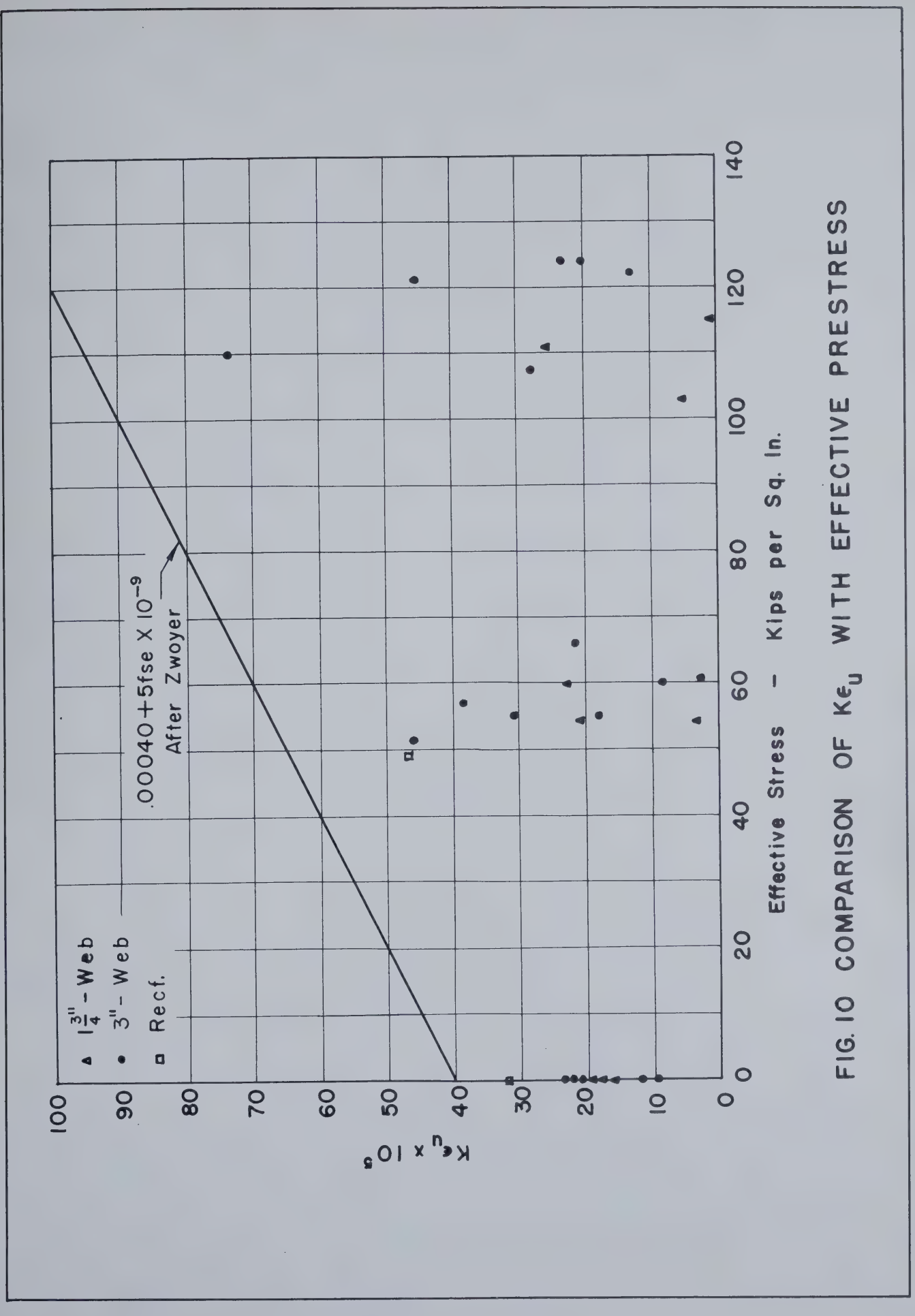


FIG. 10 COMPARISON OF $K\epsilon_u$ WITH EFFECTIVE PRESTRESS

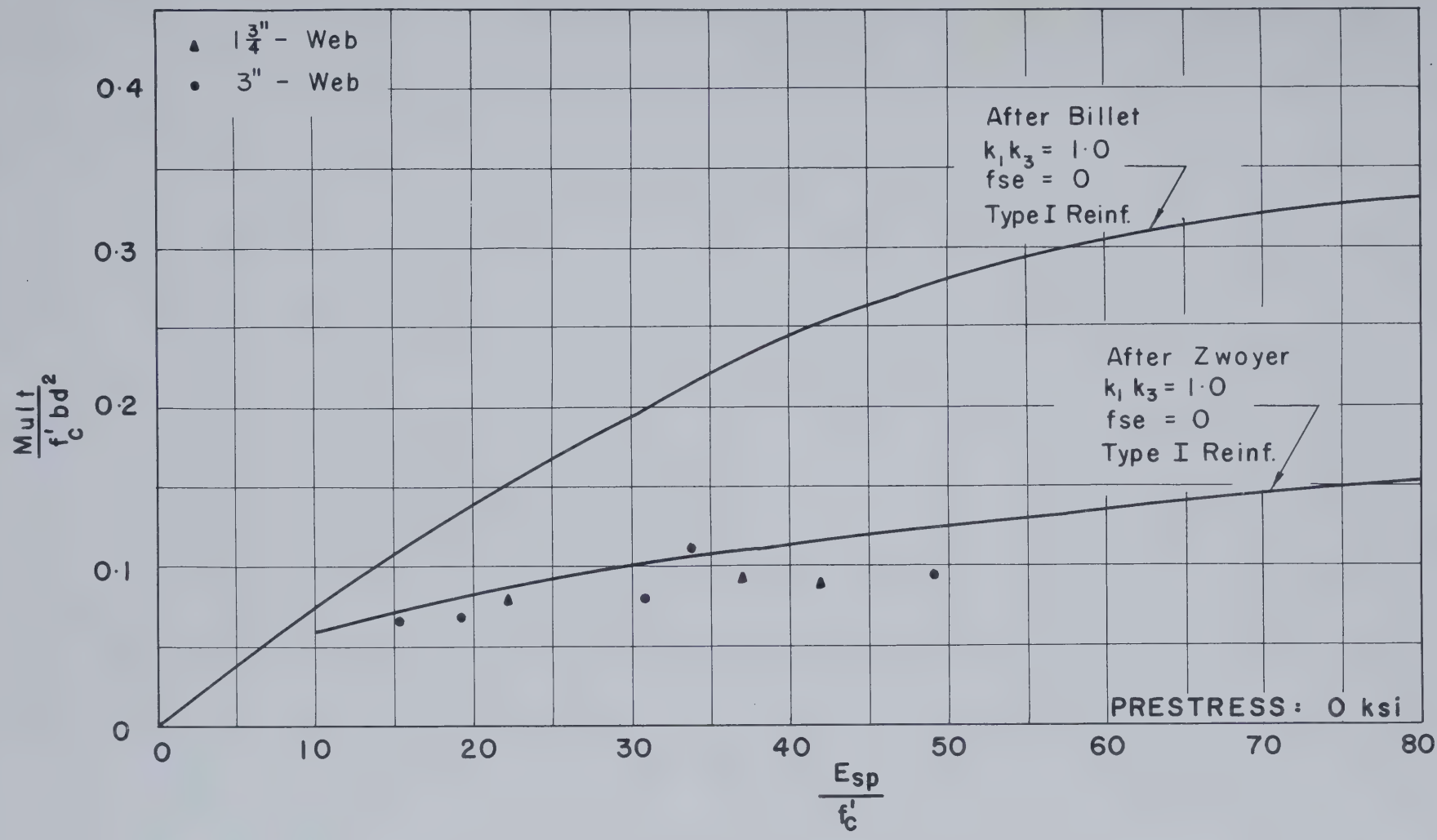


FIG. 11a COMPARISON OF SHEAR STRENGTH OF I-BEAMS WITH THEORETICAL RELATIONSHIPS FOR THE ULTIMATE SHEAR & FLEXURAL STRENGTHS OF RECT. BEAMS

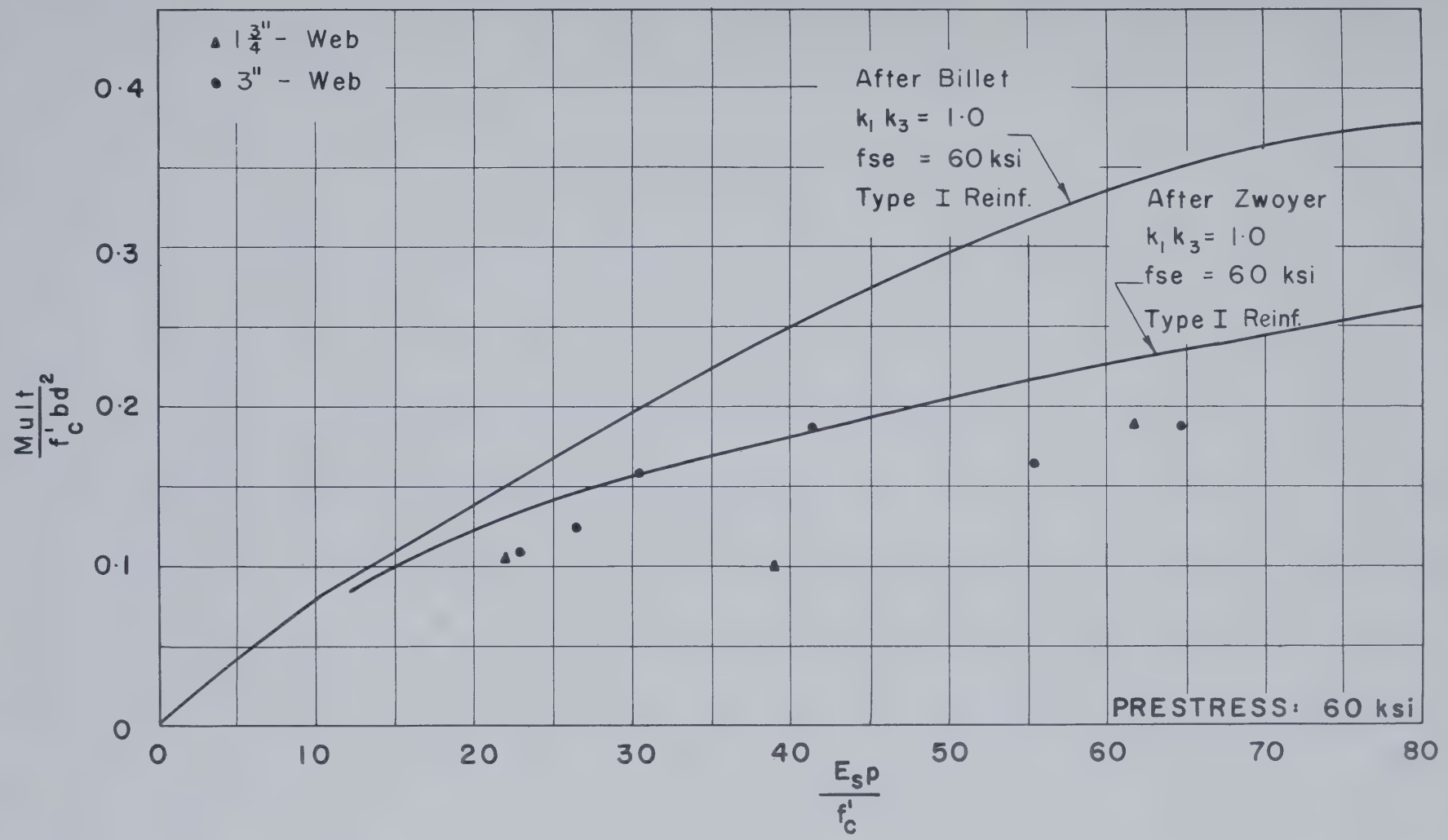


FIG. 11b COMPARISON OF SHEAR STRENGTH OF I-BEAMS WITH THEORETICAL RELATIONSHIPS FOR THE ULTIMATE SHEAR & FLEXURAL STRENGTHS OF RECT. BEAMS

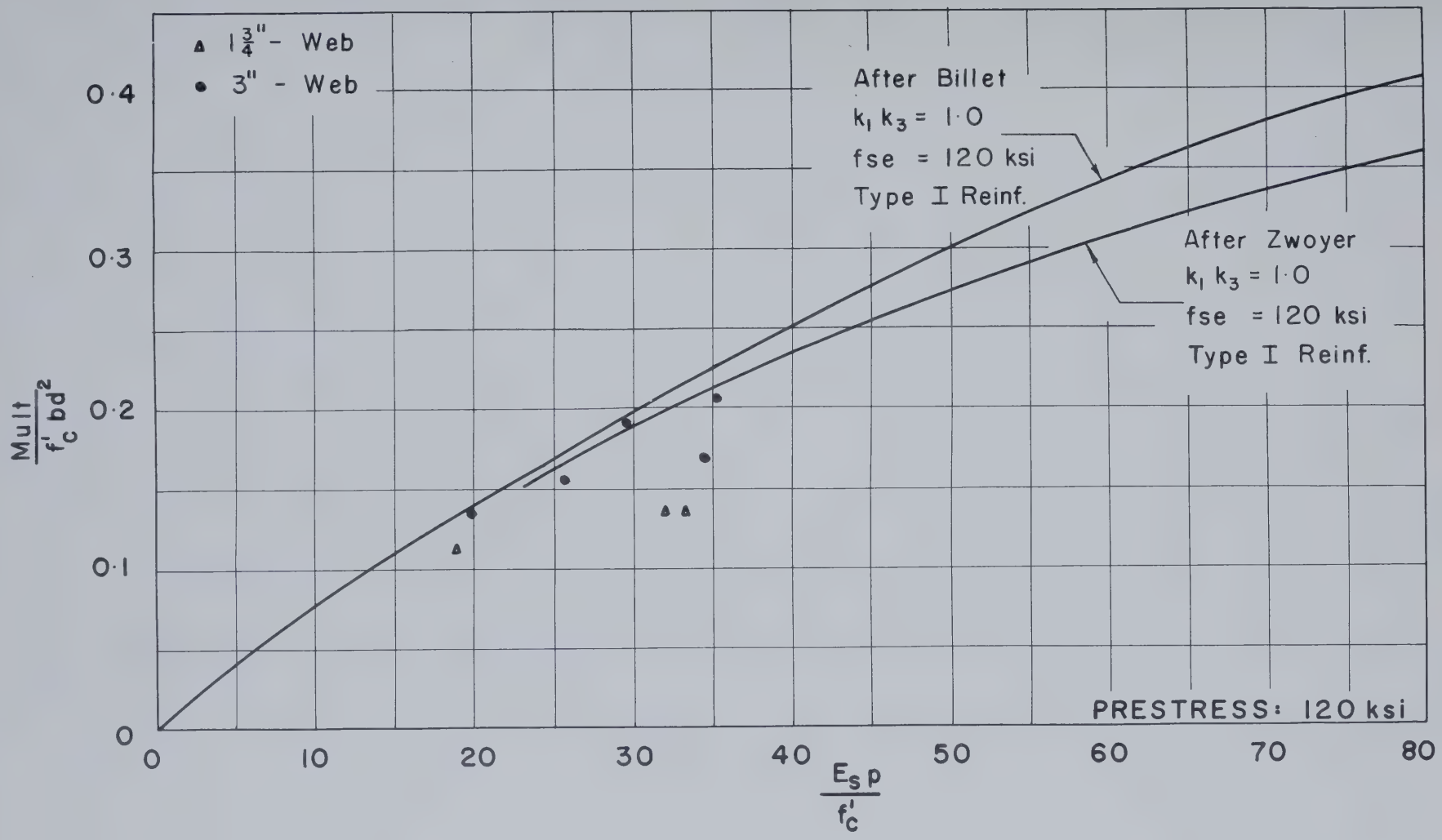


FIG. 11 c COMPARISON OF SHEAR STRENGTH OF I-BEAMS WITH THEORETICAL RELATIONSHIPS FOR THE ULTIMATE SHEAR & FLEXURAL STRENGTHS OF RECT. BEAMS

Part D

PROGRESS REPORT

ON

A STUDY OF THE COMPUTED EFFECTS OF CREEP, SHRINKAGE, AND RELAXATION ON TEST BEAMS

by

G. McLean

D. A STUDY OF THE COMPUTED EFFECTS OF CREEP, SHRINKAGE, AND RELAXATION ON TEST BEAMS

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D. A STUDY OF THE COMPUTED EFFECTS OF CREEP, SHRINKAGE, AND RELAXATION ON TEST BEAMS

I. INTRODUCTION

A series of sustained load tests on prestressed concrete beams and auxiliary specimens has been initiated in order to check the validity of the theoretical expression presented in McLean's thesis and proposed for calculating the total loss in prestress resulting from the effects of concrete creep and shrinkage, and steel relaxation. In connection with these tests, theoretical computations of losses in prestress force were made utilizing these expressions. These computations are presented graphically in this report in Figs. 1-5. The computations were made for the beams used in the sustained load tests and therefore are applicable only to beams having that section and percentage of steel. Theoretical curves are presented for the beams under dead load and prestress only.

II. DESCRIPTION OF SPECIMENS

The beams used in the sustained load tests, and for which the computations presented in this report have been made, are 4 by 6 in. in cross-section, 7 ft. long, simply-supported on a 6 ft. span. The initial specimens are post-tensioned and grouted. The beams are reinforced with five 0.192-in. diameter wires extending in a straight line between the ends of the beam. The wires are stressed to about 166,000 psi in the beam for which the theoretical curves are presented. The wires are located so that under prestress plus dead load, the stress will be a maximum at the bottom and near zero at the top.

The creep and shrinkage properties of the concrete in the beams are being measured by concurrent tests on 4-in. diameter, 16-in. long cylinders. The creep specimens are spring-loaded to a stress approximating the maximum concrete stress in the beam. Both beams and cylinders will undergo the same variations in humidity and temperature.

It is recognized that creep is primarily a dead-load problem. However it is planned to test beams in pairs, one beam having the maximum allowable compression stress in the concrete at the top fiber, and the other having this stress at the bottom fiber. Such tests will present the extreme conditions, and hence will be most valuable in appraising the applicability of the theoretical expression for the losses. The theoretical curves will be used in designing pairs of beams with the total prestress loss resulting from varying proportions of the individual effects.

III. RESULTS OF ANALYSIS

Utilizing Eq. (6) from Chapter VII of McLean's thesis and the properties of the beams described in Chapter II of this report, an analysis of the losses in prestress resulting from creep, shrinkage, and relaxation has been made for the beams under dead load and prestress only. The results of the analysis have been plotted and are presented in Figs. 1-5. These curves indicate the theoretically predicted total loss in prestress that may occur in the beams, under varying conditions of steel relaxation, and concrete creep and shrinkage. In the figures the ordinates are percent loss in prestress force and the abscissas are the creep function ϕ_t at time t. Each figure is for a particular final value of shrinkage strain - 0.0002, 0.0003, 0.0004, and 0.0005 for Figs. 1-5 respectively. Also each figure contains four curves for steel relaxations varying from 3 to 12 percent of the initial tensioning stress.

It should be emphasized that, since each curve is for one particular value of shrinkage strain and percent relaxation the curves are curves of predicted total loss in prestress corresponding to those particular values of shrinkage and relaxation. Thus the curves are not curves of loss versus time. However, curves of loss versus time could be constructed if values of ϕ_t, shrinkage, and relaxation were known at several different times; then Figs. 1-5 could be entered with these values and the total losses obtained plotted against time.

IV. SUMMARY

The results of an analytical study of the losses in prestress in test beams are presented in this report. The computations are based on the theory presented in McLean's thesis. The results are given in the form of plots of the theoretically predicted total loss in prestress under varying conditions of steel relaxation, and concrete creep and shrinkage. The results presented were computed for the test beams under dead load and prestress only.

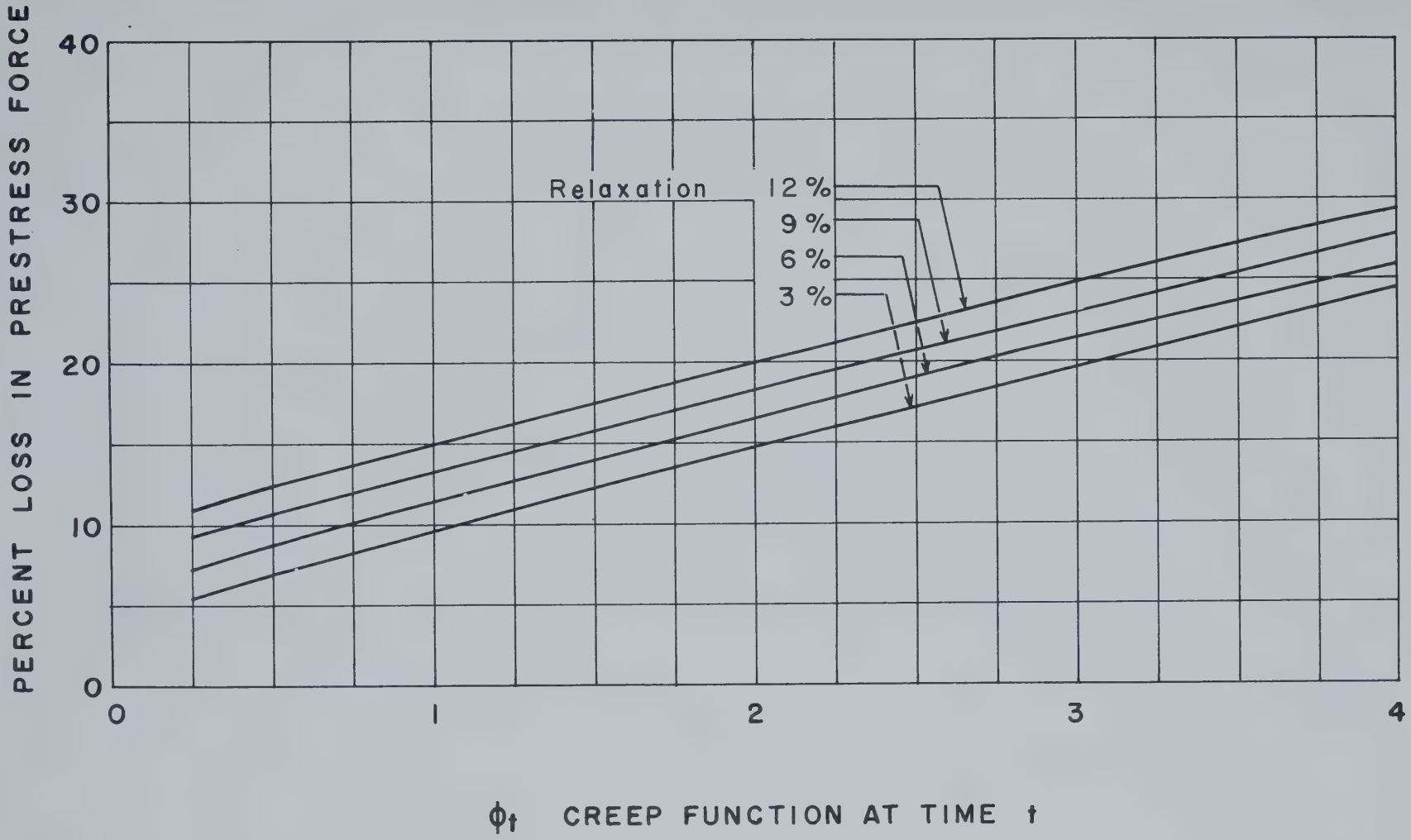

FIG. 1 PERCENTAGE LOSS IN PRESTRESS: SHRINKAGE STRAIN = 0.0002

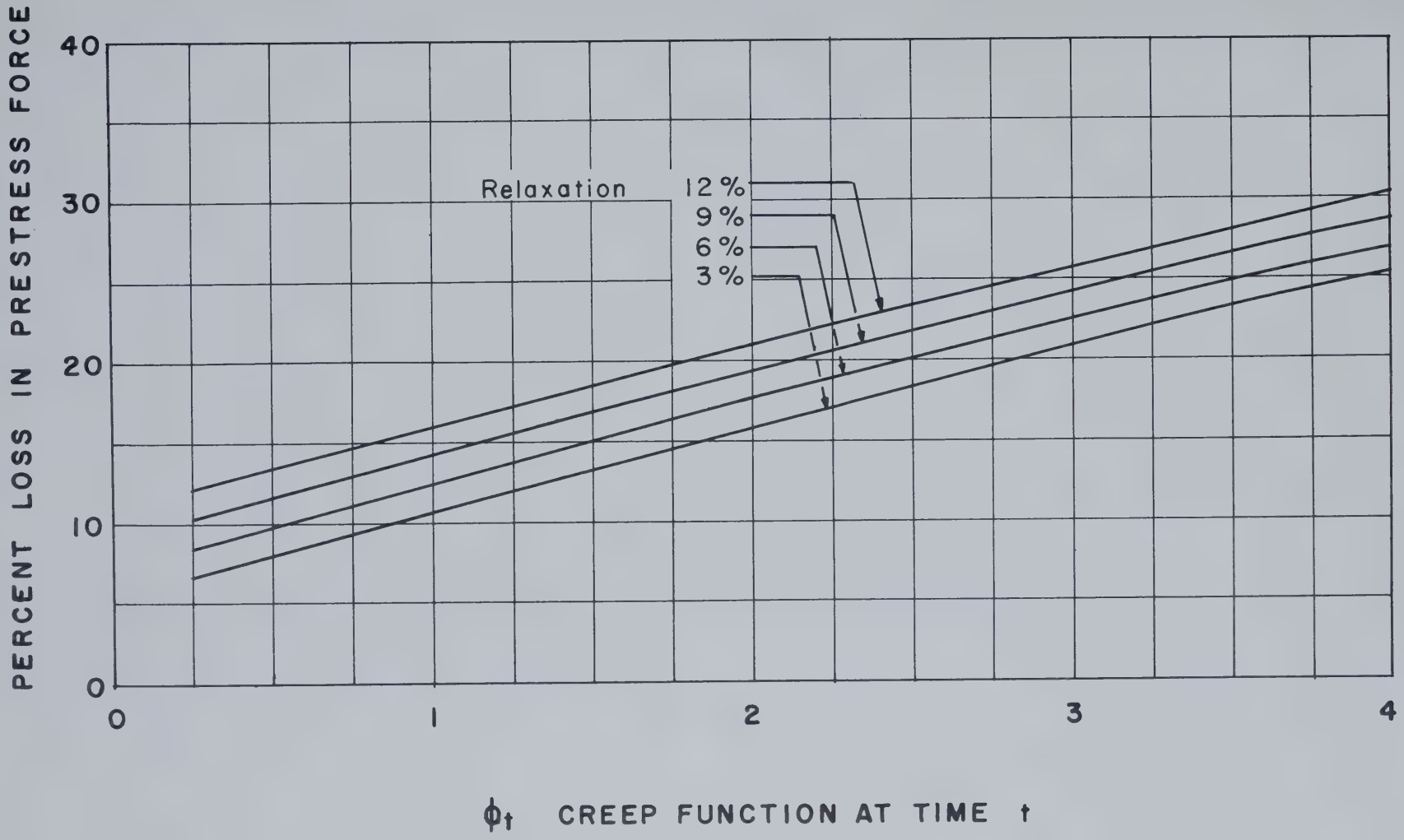

FIG. 2 PERCENTAGE LOSS IN PRESTRESS: SHRINKAGE STRAIN = 0.0003

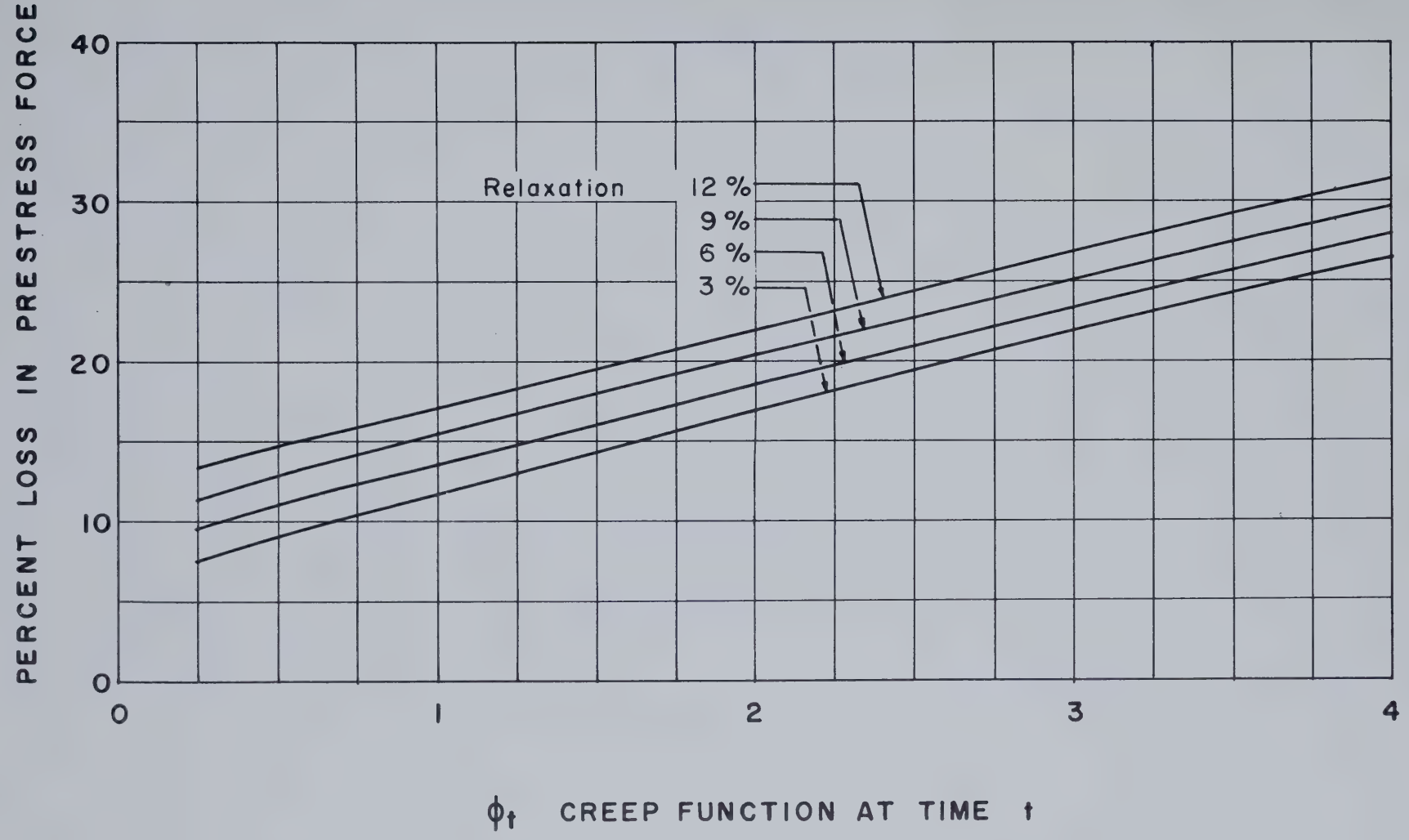

FIG. 3 PERCENTAGE LOSS IN PRESTRESS: SHRINKAGE STRAIN = 0.0004

DT.

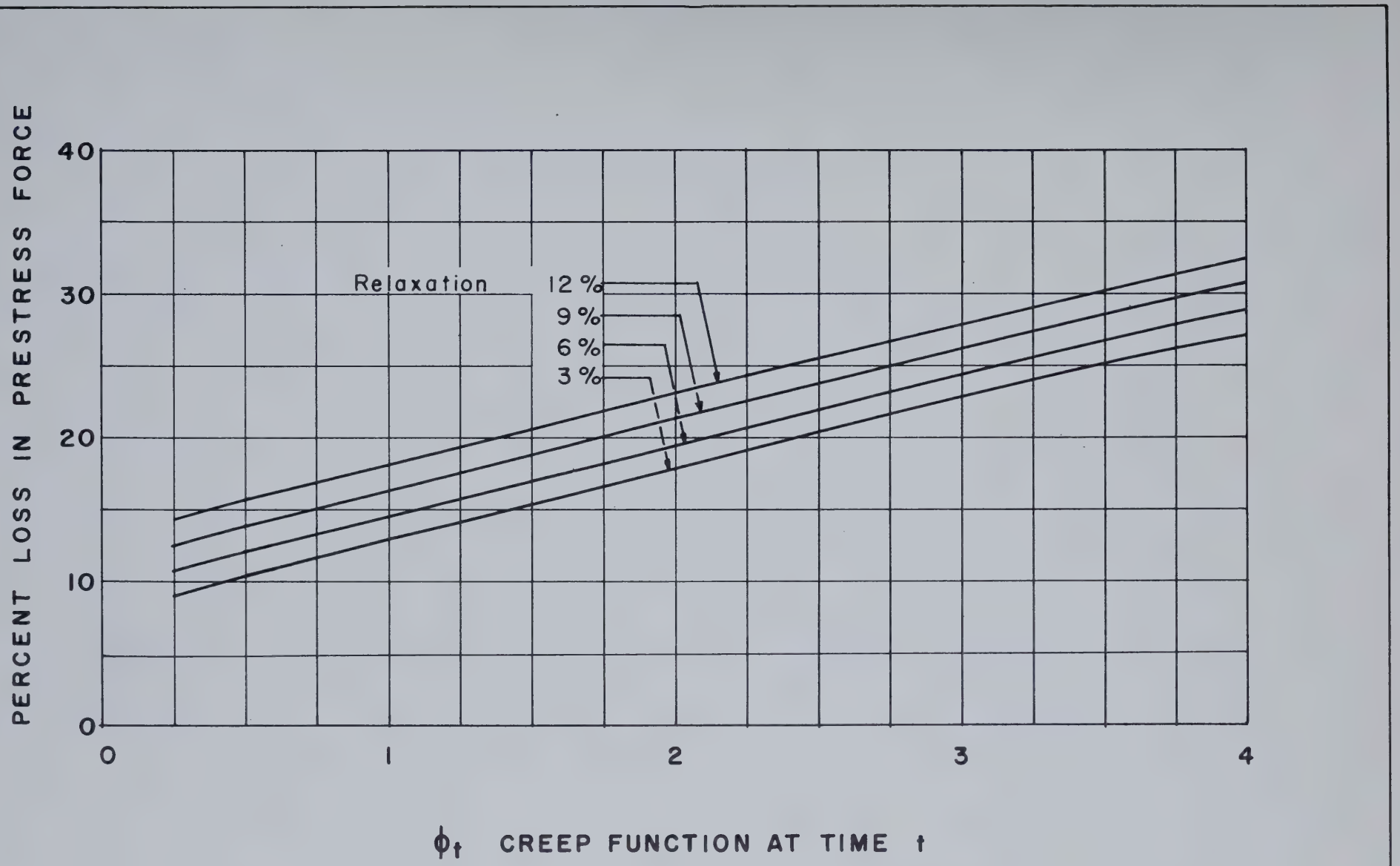

FIG. 4 PERCENTAGE LOSS IN PRESTRESS: SHRINKAGE STRAIN = 0.0005

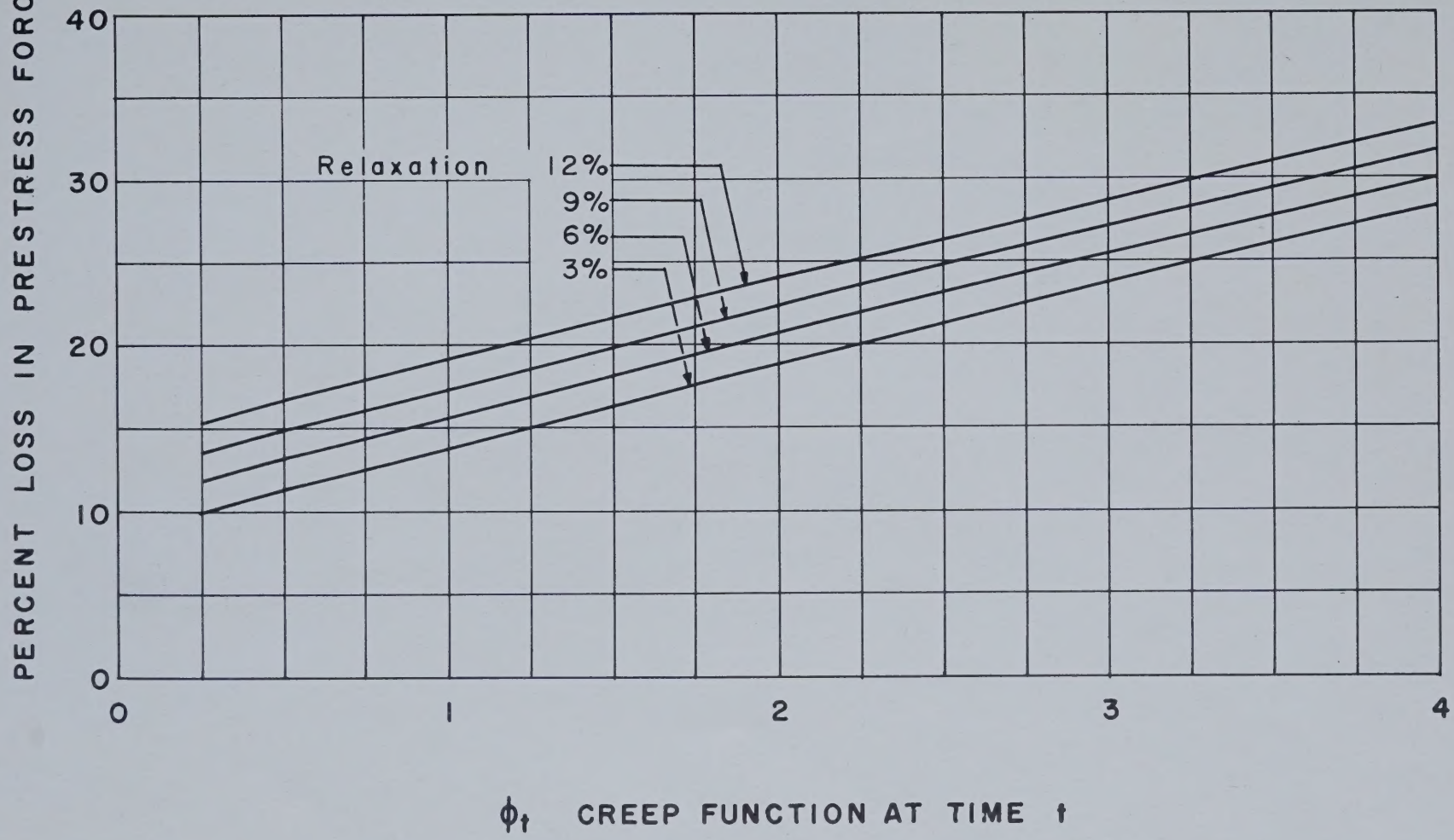

FIG. 5 PERCENTAGE LOSS IN PRESTRESS: SHRINKAGE STRAIN = 0.0006

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